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THE INVESTIGATION OF A HORIZONTALLY
STIFFENED CONNECTION

Submitted to:
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By:
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October 3, 1955

A B S T R A C T

1. Purpose

The purpose of this test is to determine the structural behavior of a horizontally stiffened, direct welded, 2-way, beam-column connection so that a basis for stiffener design may be developed.

2. Scope

The investigation is limited to the test of one connection formed by two 16WF36 beams welded to a horizontally stiffened 8WF31 column. This test is one of a series that is presently being conducted at Lehigh University.

3. Results

The results of this test can be summarized briefly as follows:

- (1) The connection develops full working load before any local yield in the connection region occurs.
- (2) Full rolled x-section plastic moment is developed.
- (3) Failure occurs in the beams at a critical section 6" from the column face.
- (4) The 7/16" thickness of stiffener is adequate to develop full beam plastic moment without stiffener deformation.
- (5) Web participation extends about $.125d$ beyond the stiffeners on each side.
- (6) The condition of the column is not critical either at column working load or overload.

I. INTRODUCTION

The adaptability of welding to continuous or "rigid" frame steel design has revolutionized the trends in modern building construction. Clear spans once thought impossible are now being spanned economically. Connection details once considered indispensable are now not used. The architect, given a greater latitude in applying his creative genius, has created structures of far greater aesthetic appeal. These advantages and many others obtained through welding do not exhaust all of the benefits that may be reaped. Time, experience, and research continually open the doors to new opportunities.

Probably the most direct way to gain more knowledge is through research. One of the major research projects on continuous design in steel structures has been carried on since before World War II at Lehigh University. The results of investigations have been published in numerous "Progress Reports on Welded Continuous Frames and Their Components".¹⁻⁸ Recently, a new project has been initiated at Lehigh University to study restraining beam-column connections. This study will include both the direct welded and seat angle or bracket with top plate types of connections.

In tier building construction there occurs a considerable repetition of beam-column connection details. The two above mentioned connection types are competitive in this kind of construction. It is the intention of the investigators to provide design criteria on both connection types from which a design procedure may be established.

The investigation will first consider the direct welded type of connection. A series of 11 tests has been proposed. This report concerns itself with the second test in this series which was on a direct welded connection using horizontally welded plates to stiffen the connection area. The scope of this report is further limited, as is the parent project, to the study of structures under static load. A complete statement of the project may be found in the Fritz Laboratory Library, File #233.

The specific objective of this test is to determine the behavior of a direct welded, horizontally stiffened, beam-column connection including the following aspects:

- (a) Elastic stress distribution
- (b) Elastic and plastic strengths
- (c) Moment-rotation relationships
- (d) Moment-deflection relationships
- (e) Column overload capacity
- (f) Stiffener and column web behavior.

With an accurate analysis of the above characteristics, it is hoped that a design procedure for horizontally stiffened connections can be recommended.

II. TEST PROGRAM

1. Test Specimen

The test program is limited to one test of the connection shown in Fig. 1. It is a stiffened, direct welded, 2-way beam-column connection. The beams are 16WF36's while the column is an 8WF31. The horizontal stiffeners are 7/16" plates butt welded to the column flanges and fillet welded top and bottom to the column web. The beam to column connection is made by butt welds on the beam flanges while the web is fillet welded on both sides. The web corners are flame cut to prevent stress concentrations.

2. Tests to be Made

As previously stated, this test is one of a series that will be conducted in Fritz Laboratory at Lehigh University under project #233. The specimens to be tested are given in Table 1. The specimens in this series are of three kinds: those without stiffeners, those with horizontal stiffeners, and those with vertical stiffeners. All are direct welded, 2-way, beam-column connections.

3. Preparation of Specimen

The preparation of the specimen started, after its fabrication, with basic measurements of its components for use in determining accurate cross sections. Positions for rotation gage bars were laid out and then remeasured after the bars were welded in place with respect to the load points and column flanges (Fig. 2).

Series	Test No.	Column Size	Beam Size	Stiffening	
				Type	Dimensions
A	1	8WF31	16WF36	None	---
	2	8WF67	"	"	---
	3	12WF40	"	"	---
B	1	8WF31	"	Fig. 6B	3.5 x 7/16
	2	8WF67	"	"	"
	3	12WF40	"	"	"
C	1	8WF31	"	Fig. 6C	7.1 x 5/16 x 22
	2	8WF67	"	"	7.2 x 1/2 x 22
	3	12WF40	"	"	10.9 x 5/16 x 22

Table 1 - Program Of Restrained Beam Tests

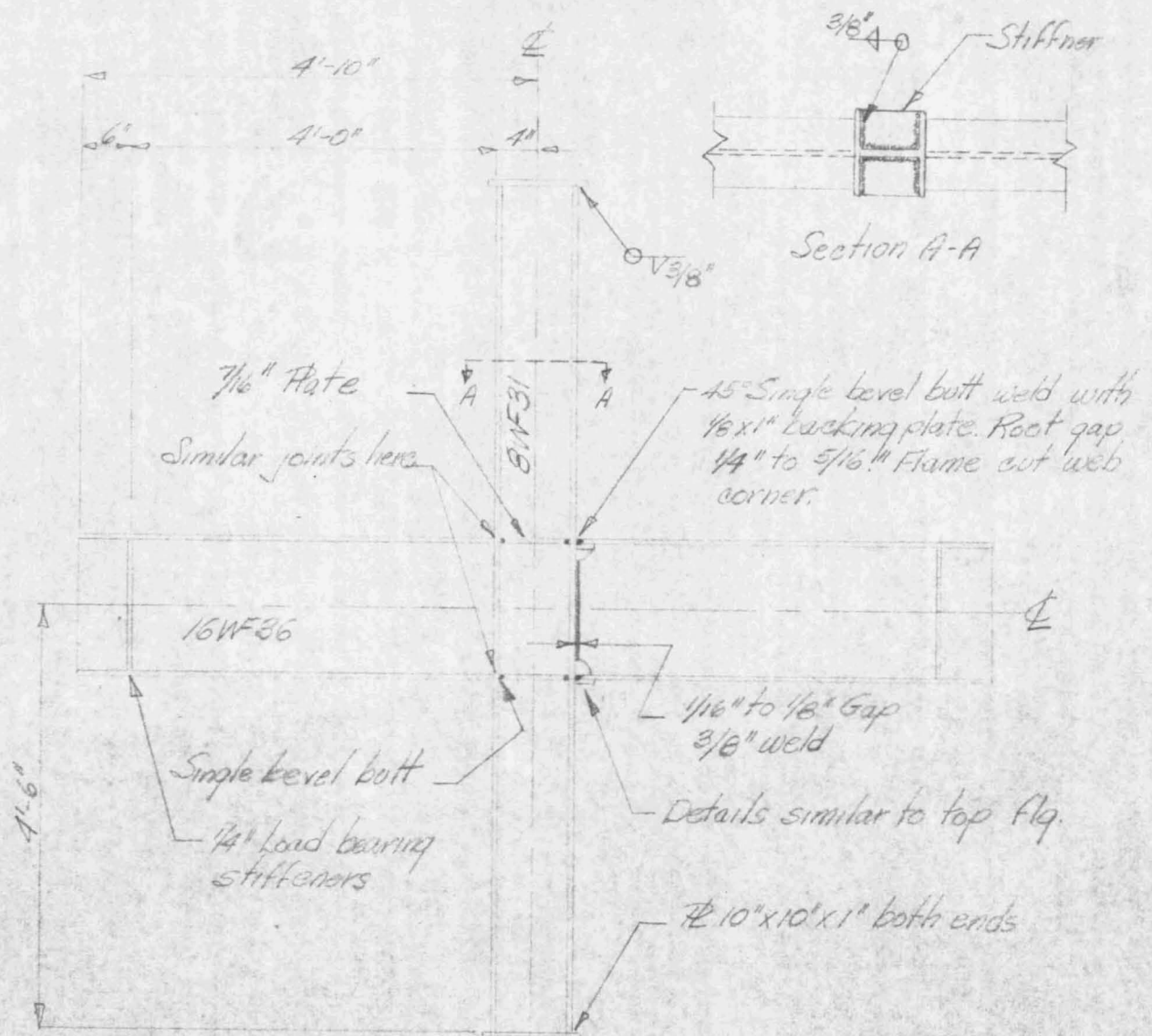


Fig. 1 The test specimen.

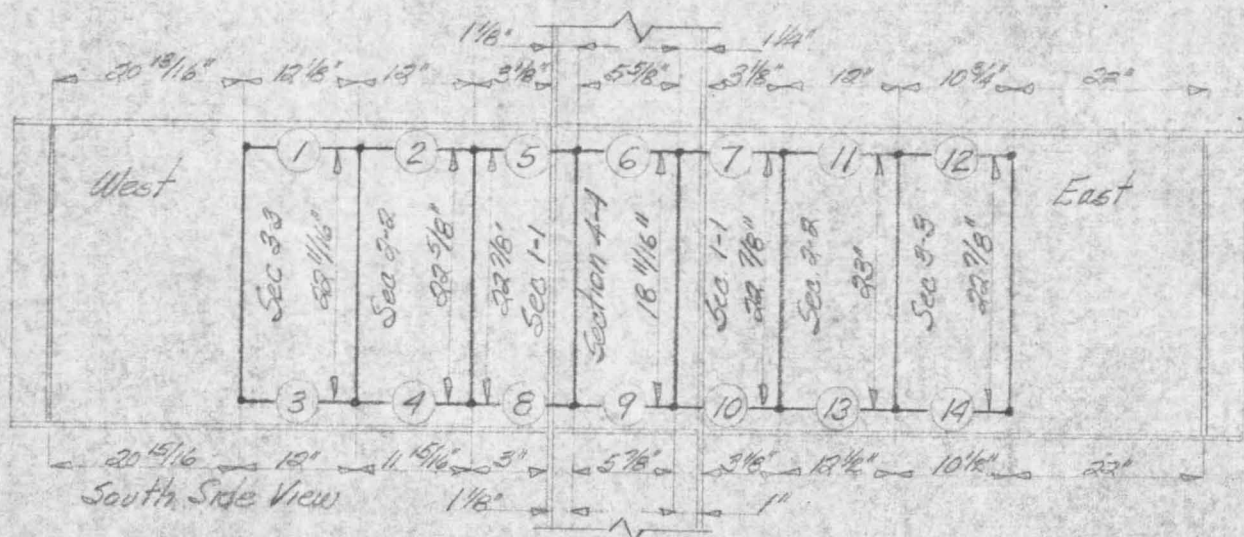


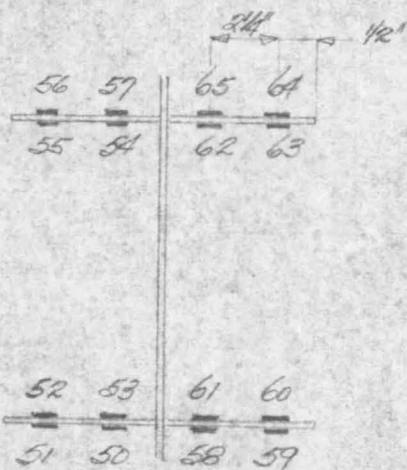
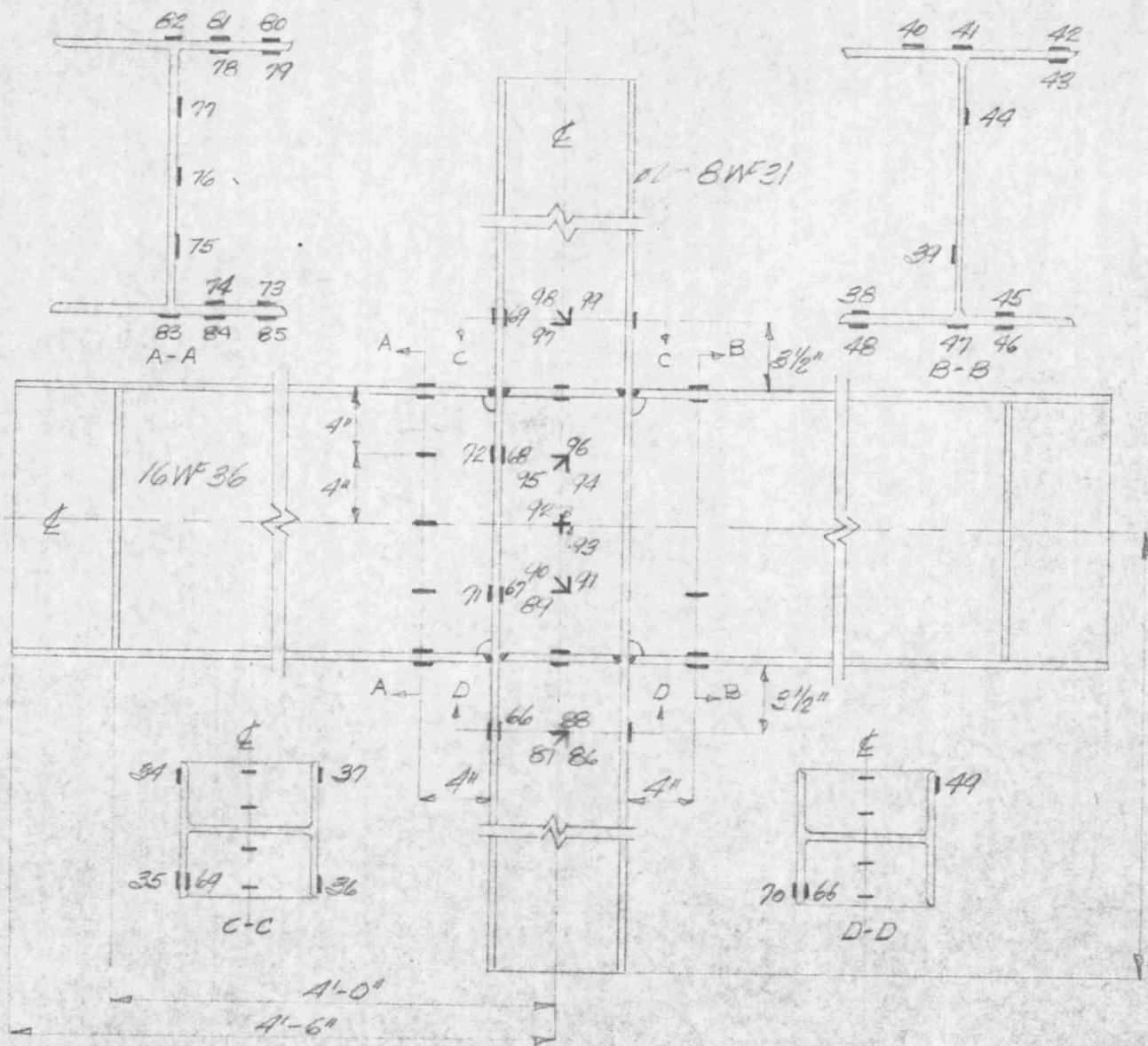
Fig. 2 Rotation gage plan for M-φ curve data.

The SR-4 gages were applied by technicians according to a gage plan supplied to them (Fig. 3). The test specimen used was the one used in test #1 (see Table 1) with the addition of horizontal stiffeners. Except for the SR-4's located on the column web, most every gage used on test #1 was in good condition and was used over in test #2. Due to the fact that it was not certain as to what gages were necessary for test #1, there were several extra gages used to insure complete coverage. Since these gages had not been analyzed before test #2, they were left on and read for test #2.

After the gages were applied the specimen was punched as indicated in Fig. 4 for flange buckling readings. The punch marks were then covered with tape and the specimen was whitewashed. The bar frame for the rotation dials was assembled and the dials were set in place. The measurements of the rotation bar frame were then taken.

4. Fabrication

Both the columns and beams were ordered in nine foot lengths. The beams were flame cut in half giving $4\frac{1}{2}$ ' length. The original ends of the beams were then prepared for butt welds. The beams were butt welded exactly at mid-column length. The stiffener plates were then added at the beam flange levels. Load bearing stiffeners were inserted at 4' from the column face. Finally, one inch thick square bearing plates were welded onto the ends of the column. All welding and cutting was done by experienced welders under the guidance of project #233 personnel.



Note:

The SR-4 gage numbers correspond to those found in the data book on file in Fritz Lab (File 233/C4) and those used in calculations in Appendix II.

Fig. 3 The SR-4 gage plan.

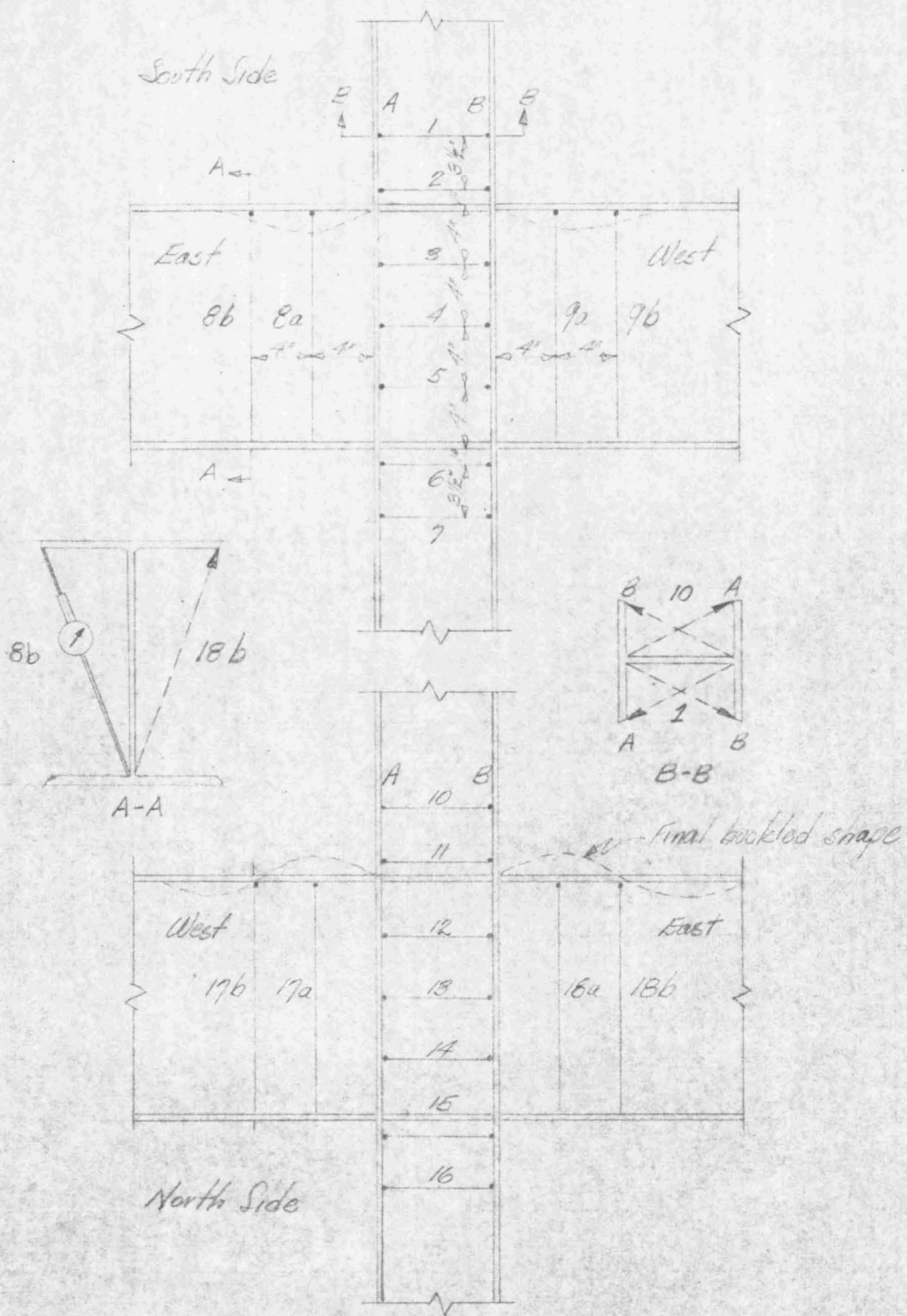


Fig. 4 Flange buckling measurements.

5. Test Setup

The test setup for test #2 was arranged so that the load application would closely approximate the actual load conditions encountered in a typical rigid frame building. To accomplish this a very simple arrangement was used. The connection was inverted and the column load was applied by the testing machine head. Fifty ton jacks were used to apply loads on the beam ends at 4 feet from the column face. The jacks were set on dynamometers which in turn rested on the machine table. There were no lateral supports at any point (see Fig. 5).

6. Measurements

Dials were used to measure beam rotations through three sections on each beam and the column centerline. These gages were mounted on bar frames which were attached to short bars of 1/2" diameter welded perpendicularly to the beam and column webs. Column and beam flange deflections were each read with a separate portable dial gage with extension. Lateral deflection of the column was measured by a fixed dial gage (mounted on a frame attached to the testing machine), at the column centerline, on a tension stiffener. Beam deflections were measured at the load points by a portable dial gage with extension.

Extensive use was made of SR-4 strain gages in the connection region. Type A-1 gages were used on the beams. Types A-1, A-7, and AX-5 were used on the column. Fig. 3 shows the entire SR-4 gage layout. SR-4 gages were used to indicate

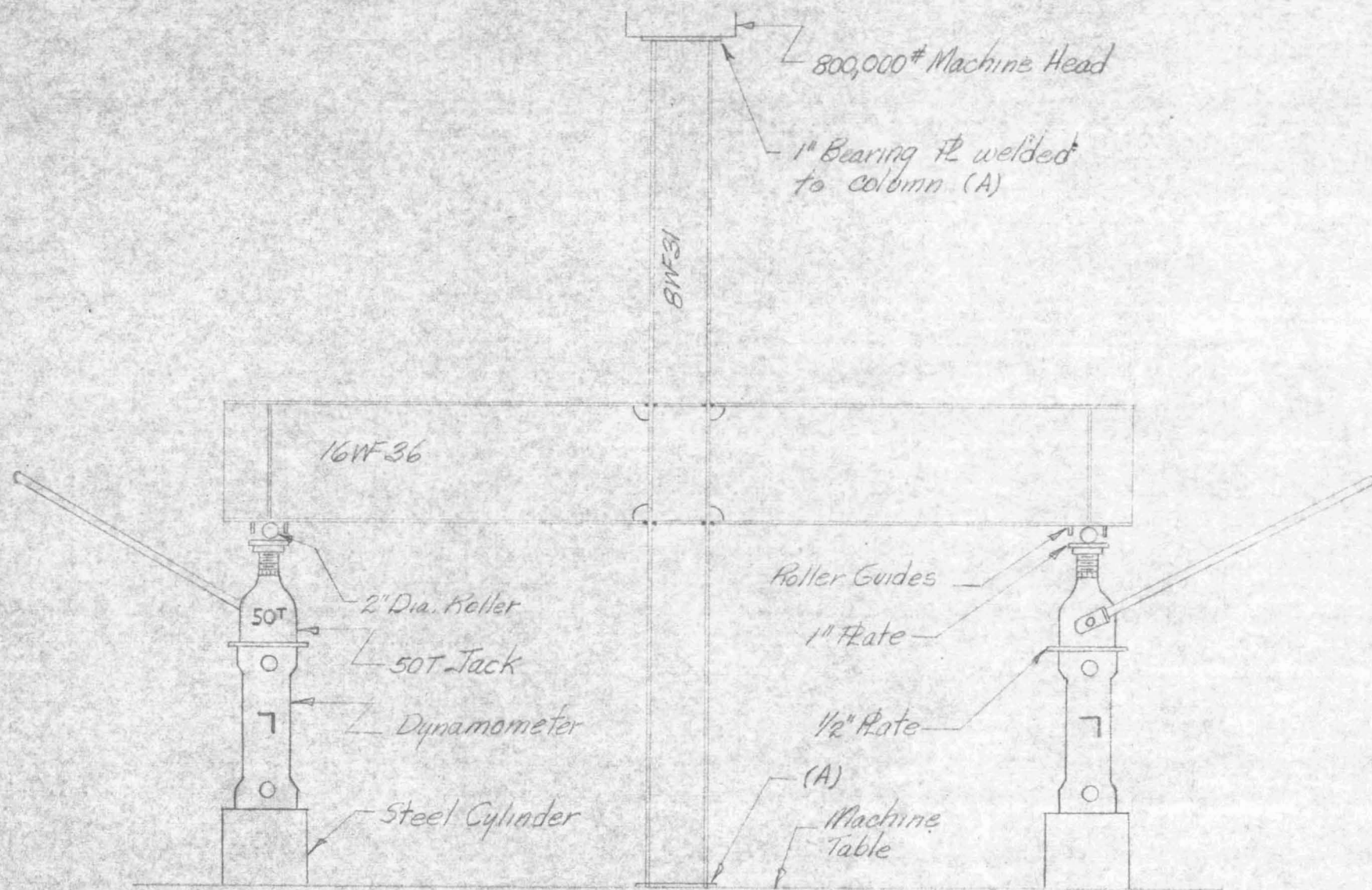


Fig. 5 The Test Setup

strain distributions within the steel; from which, stresses were then determined.

Lateral deflection of the beams was measured by a pair of buckling indicators (electrical gages) attached to the compression flange of each beam over the load point.

Whitewash was used to reveal the flaking of mill scale that occurs locally following the formation of Luder's lines at the yield point.

7. Test Procedure

The load sequence was outlined before the test. Prior to the beginning of the test, the column was checked for axial load application. Four SR-4 gages, one mounted on the outer edge of each column flange 4" above the top beam flange, were used for this purpose. The maximum gage difference compared to the 4 gage average was 7% at 38% of column working load.

The sequence of loading used can be considered to have taken place in 4 stages as follows:

1. The column load was increased in 8 equal increments to working load (132 kips) with zero load on the beams.
2. The upper column load (test position) was kept at 132 kips while a jack load was applied to each beam simultaneously in 3 kip additions until the beam working load (24 kips) was reached.

3. The column was then overloaded to 1.65 times working load (218 kips) and returned to 132 kips in 5 equal increments, beam loads remaining at a constant 24 kips.
4. The beams were then loaded by 3 kip additions until twice working load (48 kips). Deflection controlled after 48^k. The beams were loaded in odd load increments to failure.

After a beam load of 48 kips was reached an additional 3 kip increment could not be added without a large beam deflection. From this point on to failure the beams were loaded until approximately equal deflections resulted. The loads were then allowed to stabilize before readings were made. Once deflection became a control for jack load application, a criterion of 0.10 kip drift in five minutes was used as the maximum allowable beam load variation before measurements for that load increment were taken. Jack load was recorded by strain recorders which were connected to the dynamometers on which the jacks rested. The maximum jack load difference between the two jacks while deflection was used as a control was 2 kips (at last load increment).

In applying beam loads in the constant load increment range (0 to 48 kips) the jacks were pumped evenly and balanced out roughly (± 0.10 kip) after each 1 kip addition. The machine head was adjusted to 132 kips load at a jack increment of 2 kips. Then the final kip was added precisely. The final 1 kip increment did not affect the upper column load appreciably and it was not necessary to adjust further.

Control curves were kept for the load-rotation and load-beam deflection relationships for each beam during the entire test. A column lateral deflection curve was also plotted throughout the test.

The test required 28 hours for completion distributed about evenly over 3 days. There were two major shutdowns of about 12 hours duration each evening following the first and second days of testing.

The test set-up was previously described as approximating the actual load conditions found in rigid frame construction. With the knowledge of the load sequence information this statement can now be explained in detail. The column was first loaded to working load with no load on the beams. The beam working load was then added with the top part of the column (actually the column below beam level in the building) remaining at a constant 132 kips while the lower column (test position) was off loaded by twice the single beam load. In the building the beams and lower column are at working load at this point. The column was then tested for its ability to take overload at beam working load. At this point the beams are at working load (24 kips) while the lower column in a building is at overload (218 kips). The beam failure load was then determined while the lower column (building position) remained at working load.

8. The Previous Test

The specimen used in this test was, as previously stated, the same one used in test #1. During test #1 only the column was deformed in the vicinity of the compression beam flanges.

The column web was straightened by cold bending it with the aid of a testing machine. The beams themselves were not affected adversely at all in test #1. Essentially the same SR-4 layout was used on test #2 as on test #1, since 75 per cent of the SR-4 gages used in test #1 were undamaged. The mechanical gage measurements were basically the same as those used in test #1.

III. THEORETICAL ANALYSIS OF STIFFENED CONNECTIONS

1. Limitations

The theoretical curves for $M-\phi$ and $M-\delta$ relationships which appear on the curves in this report are based upon a completely rigid (fixed) end of beam condition. This is not quite the true condition, so that small deviations of the actual curves are to be expected. An attempt to establish an adjusted theoretical curve would involve making assumptions which could be questioned. These limitations and possible adjustments are discussed under the various subtopics below. The test curves though, are compared with generally accepted standards in steel design.

2. Unstiffened Connections

A short discussion at this point on the analysis of an unstiffened beam-column connection will help to point out the problems involved. As the beam load is increased in a rigid frame construction a moment is developed at the end of the beam in accordance with the per cent restraint afforded by the connection. The moment and shear can be assumed to be transmitted to the column by the beam flanges and web respectively. This simplification is not far from the actual condition. The beam flanges can now be thought of as a couple acting on the column flange. The action of the compressive thrust can cause the column to buckle. The accurate prediction of column web action for a given moment and beam size is the basic problem in an unstiffened connection.

(a) AISC Specification

Of all the specifications in current use, only the AISC specification (Sec. 26h) on the web crippling of beams seems applicable to the unstiffened column connection. The specification is for "concentrated loads not supported by bearing stiffeners" and the governing formula is:

For interior loads $\frac{R}{t(N+2k)} = \text{not greater than } 24,000 \text{ psi}$

where R = concentrated load

t = web thickness, inches

N = length of bearing, inches

k = distance from outer face of flange
to web toe of fillet, inches.

If the load R due to the compressive thrust in the compression flange is considered a concentrated load on an unstiffened beam, $R = M_b/d$, and the allowable maximum beam moment is given by:

$$M_b = t d (N + 2k) 24,000$$

Substituting into this expression the values for a 16WF36 - 8WF31 beam-column connection, a value of 230^k is obtained. Using a working stress of 20 ksi the allowable beam (16WF36) moment is 1126^k . The value 230^k is obtained by using the length of load bearing equal to the flange thickness of a 16WF36 flange (.428"). A more realistic approach would be to permit a greater bearing area because the filleted thickened section carries a stress very near that in the flanges. Also, the butt weld joining the beam flanges to the column is invariably thicker than the flange itself. Assuming a bearing value equal to k , a moment of 273^k is obtained, still only

24% of beam working moment. This specification then, does not apply.

(b) Principle Stress

One of the most desirable methods of determining critical loads upon a structure is to evaluate a stress for a given loading and compare it with an "allowable". The evaluation of a principle stress for the unstiffened web may be related to web buckling. The whitewash can be used to indicate yield stress in the web. A correlation may then be established between the principle stress at a particular place on the web and yielding of the web, thereby determining a criterion for design based upon principle stress.

(c) Plate Buckling

The analysis of an unstiffened connection may take the form of a plate buckling solution. It is just mentioned here for the case of the unstiffened connection, but the general method of attack will be presented later with regard to the buckling of the compression stiffeners.

3. Stiffened Connections

In a stiffened connection the gap between the column flanges is bridged in some manner so that the entire beam moment is no longer carried by just the column web. Probably the most common method is the use of horizontal plates butt welded at the level of the beam flanges. These plates serve two main functions. First, they transfer the thrust in the beam across the column depth directly; and second, they serve to stiffen the column web so that its own load capacity is increased.

Another method of stiffening connections is by use of vertical plates as shown in Fig. 6. Stiffening in this manner, in effect, creates three webs. Considering the web and each plate as a column whose length is the depth of the column used, the analogy of a continuously loaded beam (represented by the uniform load of beam thrust in the compression flange) over three supports can be made. This method of stiffening, though, does not offer any stiffening to the column web.

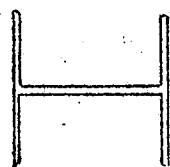
The discussions that follow will be limited to the horizontally stiffened type connections.

(a) Continuous Beam Concept

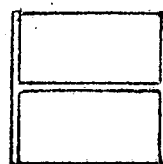
The use of horizontal stiffeners, in reality, continues the beam through the column. The "beam" formed by the stiffeners and column web differs from the beam framing into the column in several important ways. First, the plates are usually the same width as the column flange while the beams framing into the column are invariably narrower. The flanges of the connection beam are, therefore, larger. Second, the action of the column web does not stop at the top of the stiffener plates. Part of the web area above the plates undoubtedly acts to give an increased moment of inertia. Another difference is that the presence of fillet welds joining the plates to the column web also increases I , though to a much smaller degree.

With the above background the section properties for the beam through the connection can be determined (see Appendix I). The theoretical $M-\phi$ curve for this section has a smaller slope than the experimental curve has because of the larger section that actually acts. From the slope of the experimental

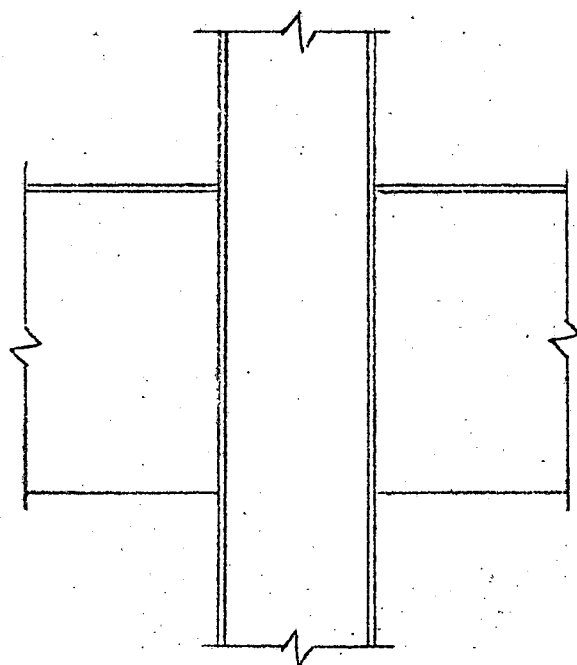
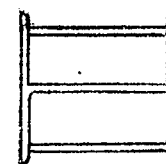
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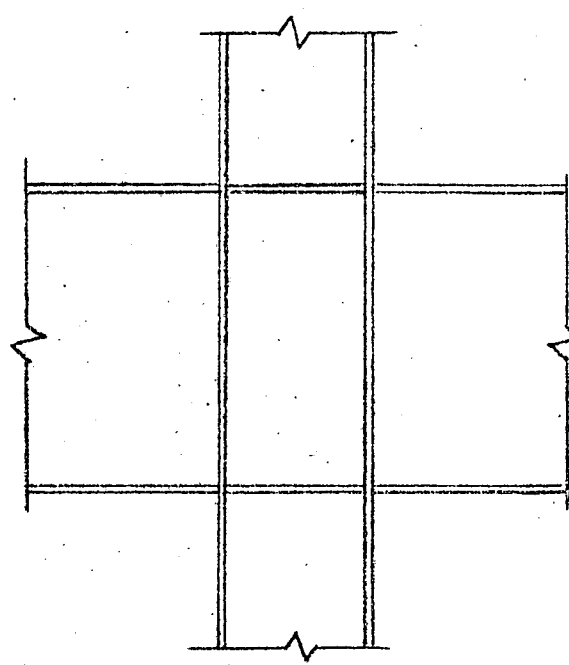
B



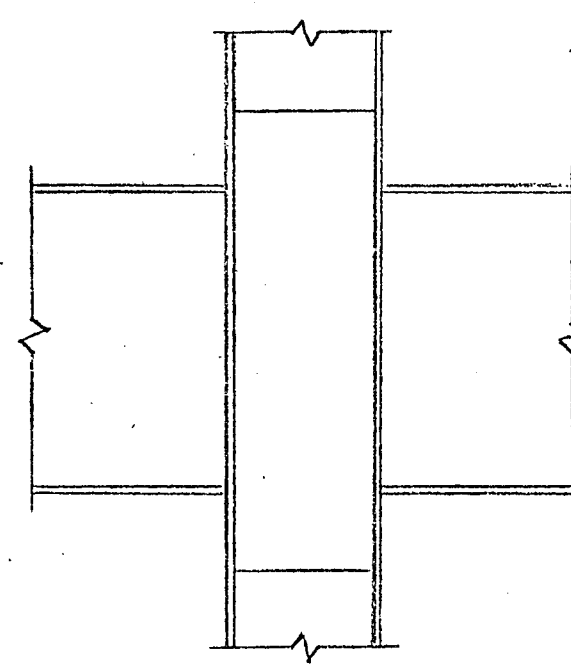
C



No
Stiffeners



Horizontal
Stiffeners



Vertical
Stiffeners

Fig. 6 Types Of Stiffening That Can Be Used

curve the amount of effective web may then be approximated (see Appendix II). This approach is not strictly correct because effect of the column flange is omitted. No doubt the flange does influence the actual $M-\phi$ curve for the center section, but the degree of influence is a question. The $M-\phi$ values for section 1 - 1 in each beam are also affected.

One of three possible situations exists with regard to the column flange effect. It is conceivable that the flange offers next to nothing in resistance to the rotation of the stiffener beam and acts basically as a beam section does. Another possibility is that the column flange acts as an incompressible layer but rotates as a plane more or less. For this case the moment arm for any section including the column flange should be reduced by $1/2 t$ (where t is flange thickness). One final possibility, especially in a heavy column, is that the flange, when stiffened by horizontal plates, offers an infinitely stiff barrier past which no rotation takes place. The results of test #1 indicate that the first stated situation is the more nearly correct one; namely, that the presence of the column flange does not materially affect the actual $M-\phi$ relationship. The web participation is calculated on this assumption.

(b) Stiffener Buckling

Once it is decided that stiffeners are needed, the questions of what kind and how much must be resolved. This test employed $7/16$ " horizontal stiffeners. The $7/16$ " thickness was arbitrarily decided on and this thickness proved to be quite adequate. The problem of stiffener size can be thought of as

one in plate buckling. The applicable formula is as follows:⁹

$$\begin{aligned} \sigma_0 = & \frac{t^2}{12(1-\nu_x\nu_y)} E_{tx} \left(\frac{\pi}{\ell}\right)^2 + E_{ty} \left(\frac{2}{b}\right)^4 \left(\frac{\ell}{\pi}\right)^2 \frac{2B + B^2C_3}{\frac{1}{3} + BC_1 + BC_2} \\ & - (\nu_y E_{tx} + \nu_x E_{ty}) \left(\frac{2}{b}\right)^2 \frac{BC_4 + B^2C_5}{\frac{1}{3} + BC_1 + B^2C_2} \\ & + 4(1-\nu_x\nu_y) G_t \left(\frac{2}{b}\right)^2 \frac{1 + BC_6 + B^2C_2}{\frac{1}{3} + BC_1 + B^2C_2} \dots (a) \end{aligned}$$

For the case of the horizontal stiffener, the loaded edges are fixed and it can be reasonably assumed that $B = \infty$. The above expression then reduces to:

$$\begin{aligned} \sigma_0 = & \frac{t^2}{12(1-\nu_x\nu_y)} E_{tx} \left(\frac{\pi}{\ell}\right)^2 + E_{ty} \left(\frac{2}{b}\right)^4 \left(\frac{\ell}{\pi}\right)^2 \frac{C_3}{C_2} \\ & - (\nu_y E_{tx} + \nu_x E_{ty}) \left(\frac{2}{b}\right)^2 \frac{C_5}{C_2} + 4(1-\nu_x\nu_y) G_t \left(\frac{2}{b}\right)^2 \frac{C_7}{C_2} \dots (b) \end{aligned}$$

The basic expression is for the condition that the plate reaches the strain hardening range. This condition will not be met unless thick plates are used. For the case of horizontal stiffeners, relatively thin plates are used. Moreover, the thinnest plates possible are sought. It is therefore safe to assume that elastic buckling occurs. For this condition the following is true:

$$E_{ty} = E$$

$$\nu_x = \nu_y = \nu$$

Hence

$$\begin{aligned} \sigma_0 = & \frac{t^2}{12(1-\nu^2)} E_{tx} \left(\frac{\pi}{\ell}\right)^2 + E \left(\frac{2}{b}\right)^4 \left(\frac{\ell}{\pi}\right)^2 \frac{C_3}{C_2} - (E + E_{tx})\nu \left(\frac{2}{b}\right)^2 \frac{C_5}{C_2} \\ & + 4(1-\nu^2)G_t \left(\frac{2}{b}\right)^2 \frac{C_7}{C_2} \dots (c) \end{aligned}$$

Substituting appropriate values into expression (c) we obtain a critical stress of 163 ksi. Calculations are shown in Appendix II.

Using expression (c) and substituting the yield stress for σ_0 the minimum thickness t for the horizontal plates is obtained. Minimum thickness calculations are shown in Appendix II.

The preceding expressions used to calculate σ_{cr} and t are not as yet developed to the point where they can be used to apply exactly to the situation of a stiffened beam-column connection. The basic assumption made in the presented calculations are that the welded sides of the plate are fully fixed, that the width of a plate includes $1/2$ the column web thickness, and that load is applied uniformly across the ends of the plate.

(c) Web Participation

The degree of web participation is an all important factor in the strength of a connection. Web participation will be determined by three methods which will be but mentioned at this time. The relation of web participation to the center of connection M- ϕ curve has already been discussed. The two other methods are based on strain distributions in beams, stiffeners, and column web. They are discussed fully in the discussion part of the report. One method is based on stiffener strain distribution, while the other is based on column web strain distribution.

IV. TEST RESULTS

The test results are arranged in sections according to component parts of the connection; namely, beams, columns, and stiffened region.

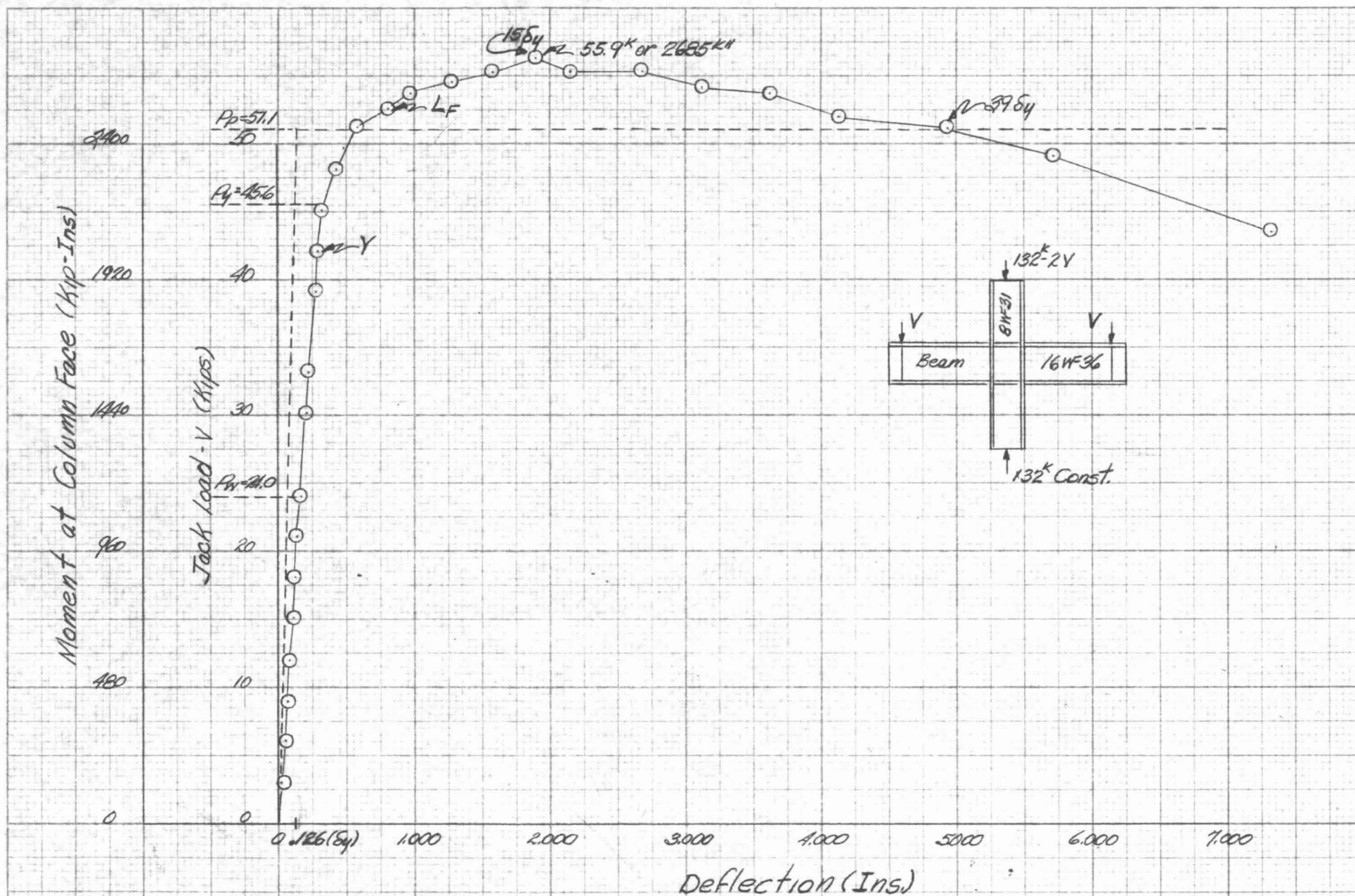
1. Beams

(a) Deflection

The most important single relationship with regard to specimen behavior is the moment-end deflection curve. This curve is shown in Fig. 7 for the West beam. It actually gives a rapid summary of the test. The curve indicates yielding at about $2160k''$ ($V = 45k$). A maximum moment of $2685k''$ ($V = 55.9k$) occurs at a deflection of $15\delta_y$. It is interesting to note that for a short cantilever length of only 4 feet, a jack load of 181% of working load is obtained at better than 7 inches end of beam deflection. The East beam reacted in almost identical manner. The final deflected shape of the beams is shown in Fig. 8.

(b) Rotation

As indicated in Fig. 2, the beam rotation measurements were taken so that $M-\phi$ curves would be obtained for three different conditions of rotation. Section 1-1 would go through full plastic rotation. Section 2-2 would have but partial plastic rotation if the jack load became great enough (which it did), and section 3-3 would be sure to stay elastic throughout the test. The $M-\phi$ relationship for sections 1-1, 2-2, and 3-3



are shown in parts A, B, and C of Fig. 9 respectively. The $M-\phi$ curve for section 1-1 is shown to an enlarged scale in Fig. 10 for the West beam. The curve indicates that even though section 1-1 suffered a rotation of $4.5\phi_y$ the load was still increasing. Fig. 11 shows the large amount of rotation and flange and web deformation that took place in the region of section 1-1. It is interesting to note in Fig. 8 that despite the large beam deformation that did take place, no rotation of the beam about its center of gravity occurred i.e. the web remained vertical.

(c) Strain Distributions

In an attempt to correlate the strains in a beam section near the column face with those in the stiffeners, a concentrated SR-4 pattern was employed at 4" from the column face on each beam. The strain distribution in a beam section of this type has been checked many times in previous investigations, but it was decided that they would be determined for at least the first test. Since the column web in test #1 failed far below σ_y for the beams, the SR-4 gages were not damaged. It was felt at the start of test #2 that all of the SR-4's used on test #1 were not needed, but that they would all be read. Most of these gages though, proved to add materially, upon being analyzed, to the general picture of the connection behavior.

The corresponding strains for a given center of connection moment (or given end of beam load) for the compression flanges of both beams and the compression stiffeners is shown in Fig. 12. The center of connection moments indicated are for M_w , M_y , and M_{2w} ; as calculated using the continuous beam concept. The strains for the south edge of the West beam are a little high

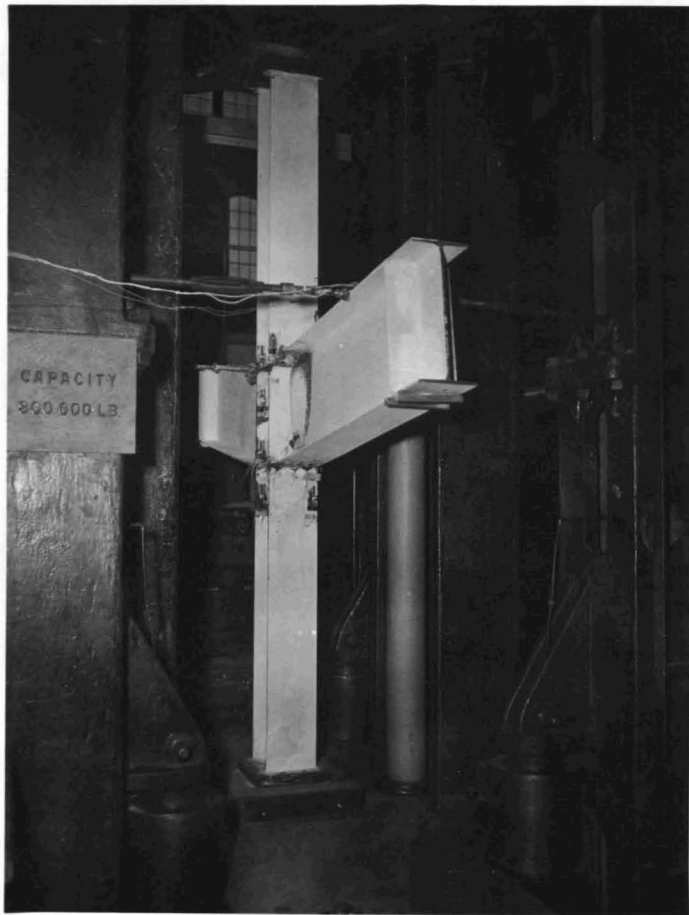
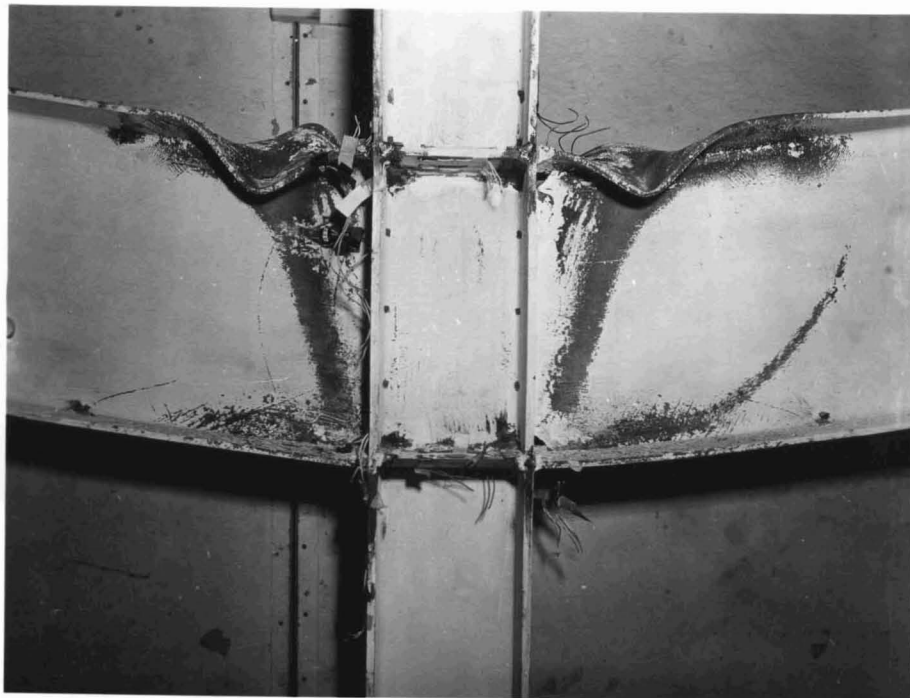


Fig. 8 Photos showing the final deflected shape of the beams. Note that beam webs and column are still vertical after test, left photo. Below, a close-up of connection region.



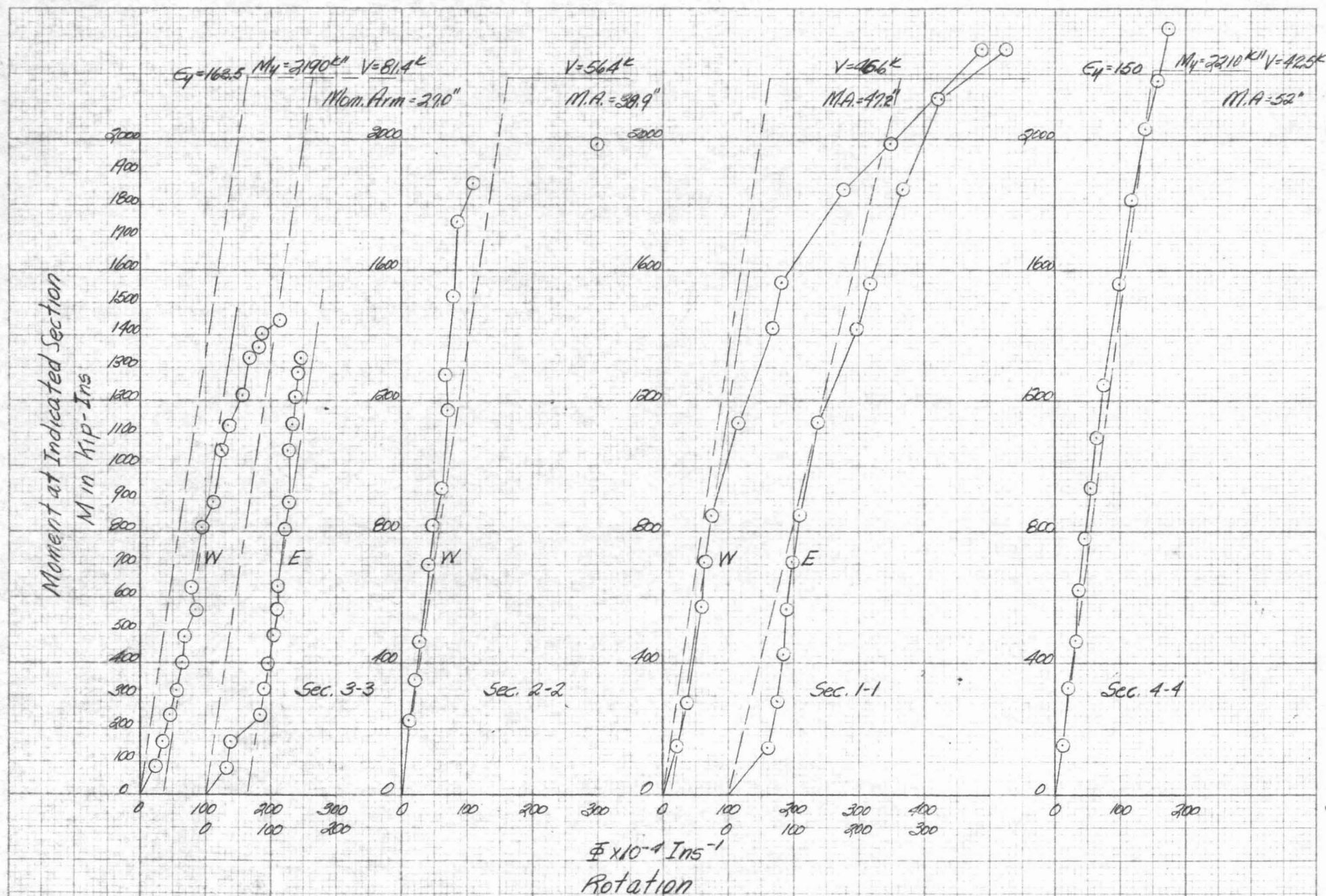


Fig. 9 Moment-Rotation Relationships For Both Beams And Center Section

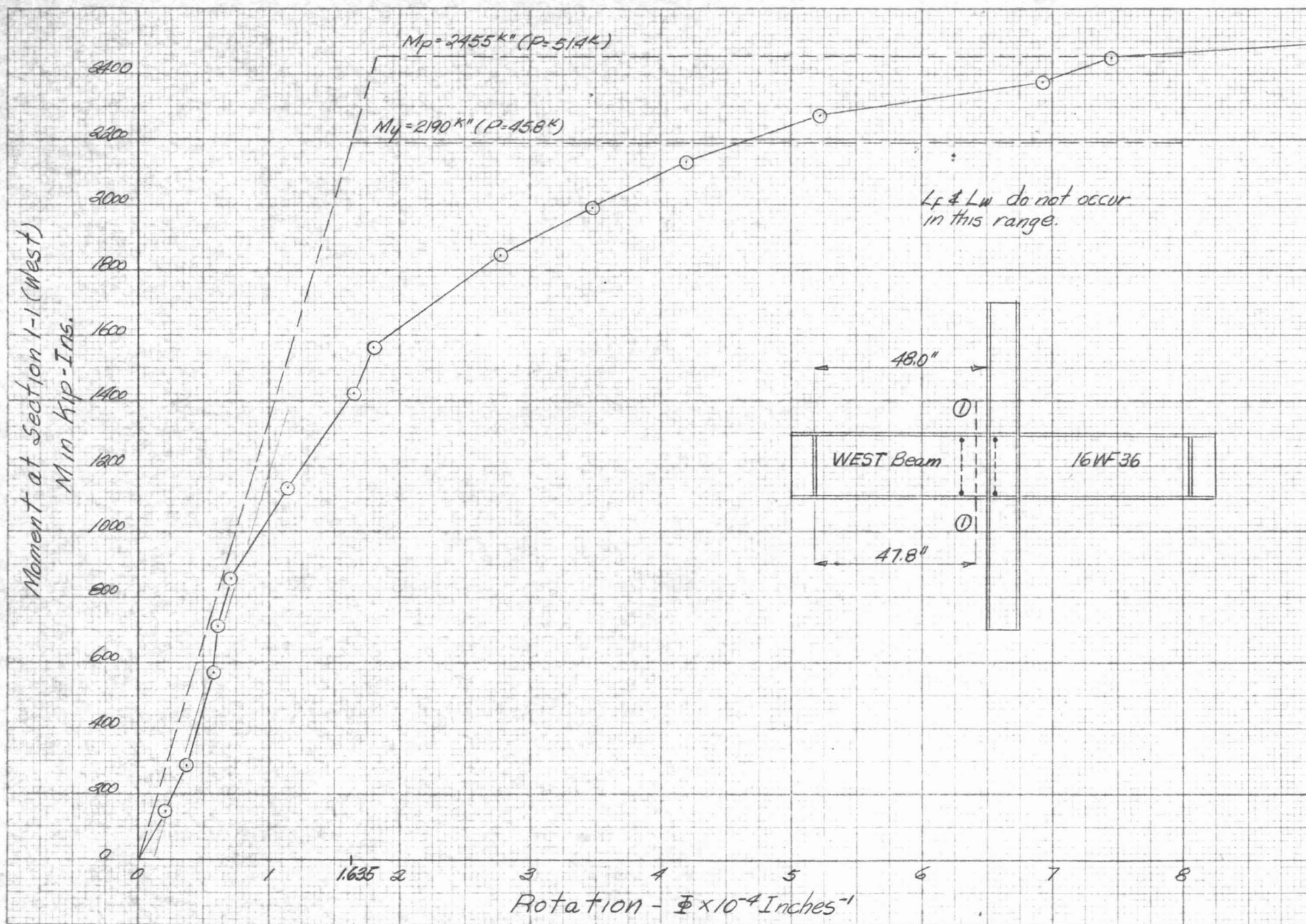
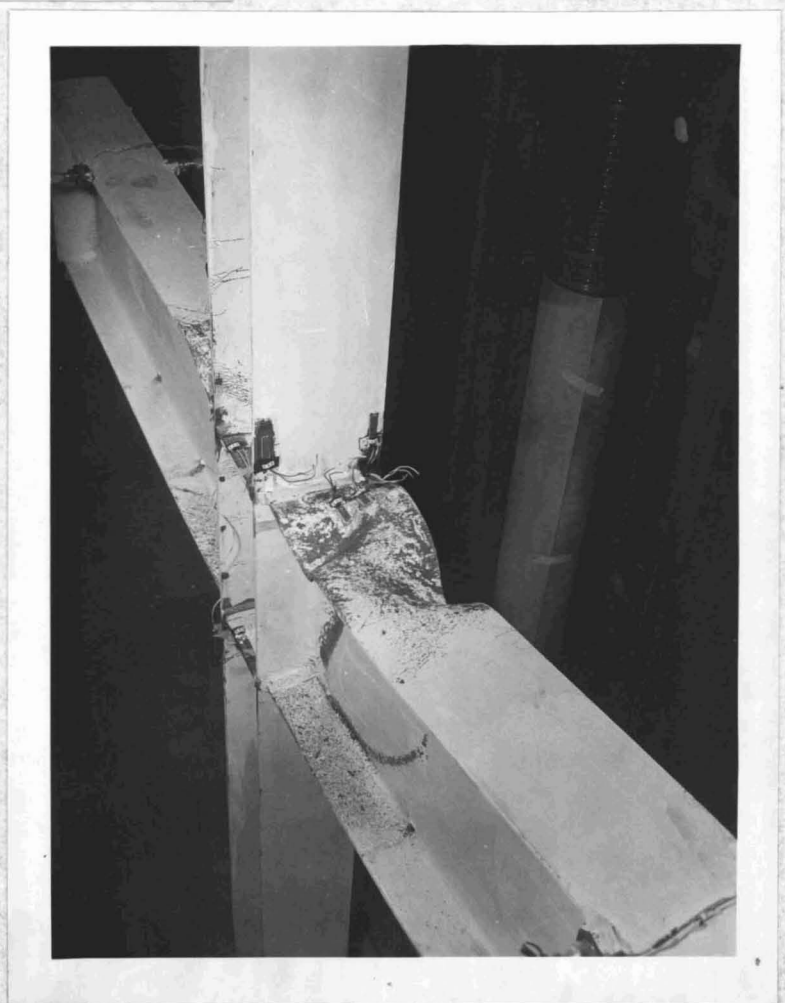


Fig. 10 Moment-Rotation Relationship For Welded Section On West Beam To A Larger Scale



Fig. 11 Photos showing the critical beam sections for the West beam, left; and for the East beam, below. Note that the column flanges are not affected.



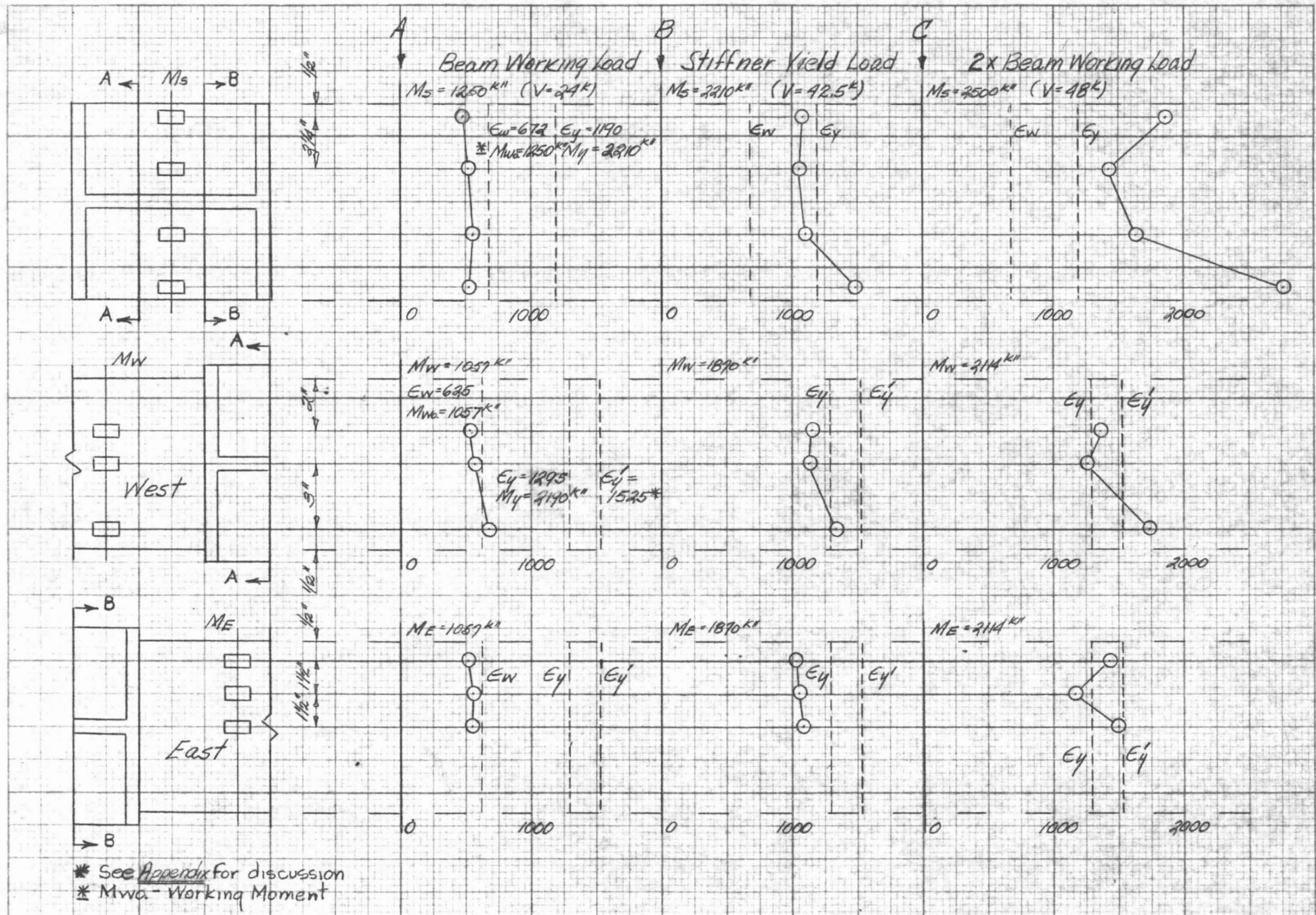


Fig. 12 Strain Distributions In Compression Stiffeners And Beam Flanges For Significant Beam Loads

and probably indicate that the West beam was slightly tilted out of the plane of the connection.

The curves for strain distributions in the beam flanges are presented in a slightly different manner in Fig. 13. It is obvious that strain deformations in the tension flanges were much greater than those in the compression flanges. The tension flange strains along the web are much greater than the edge of flange strains. The ratio of center to edge strains are in the order of 2:1 for the West beam and 7:1 for the East beam.

The strain distributions through the beam webs 4 inches from the column face are shown in Fig. 14. The actual strains at M_y are in excess of ϵ_y theoretical, especially in the tension half of the web, for both beams. These indicate stress concentrations due to welding. The strain distribution at maximum moment shows excellent correlation with the theoretical distribution usually assumed for a plastic section.

(d) Buckling

One of the best methods of determining the point at which things start to happen to the specimen is flange and web buckling measurements. These may be correlated with load data and the specific causes for sudden load changes may be pin pointed to a local failure in the specimen. Fig. 15 is a flange buckling curve for the compressive flange of the West beam at section 8b. There is a clear indication that the flange started to buckle at about 55 kips. A check of Fig. 7 shows that the maximum load was obtained at this point, after which it progressively decreased with increased end deflection. Fig. 11 confirms the buckling

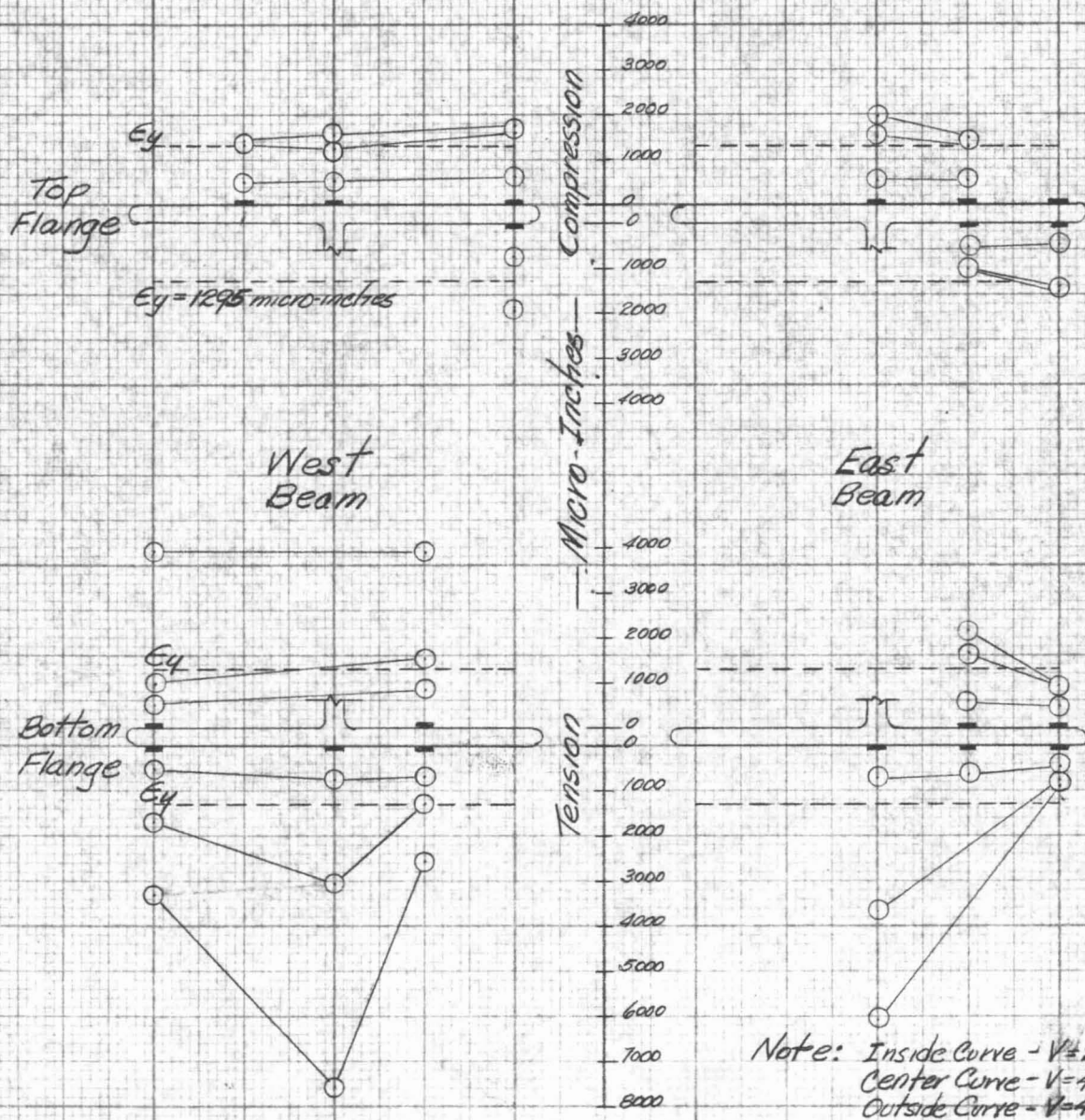


Fig. 13 Beam flange strain distributions.

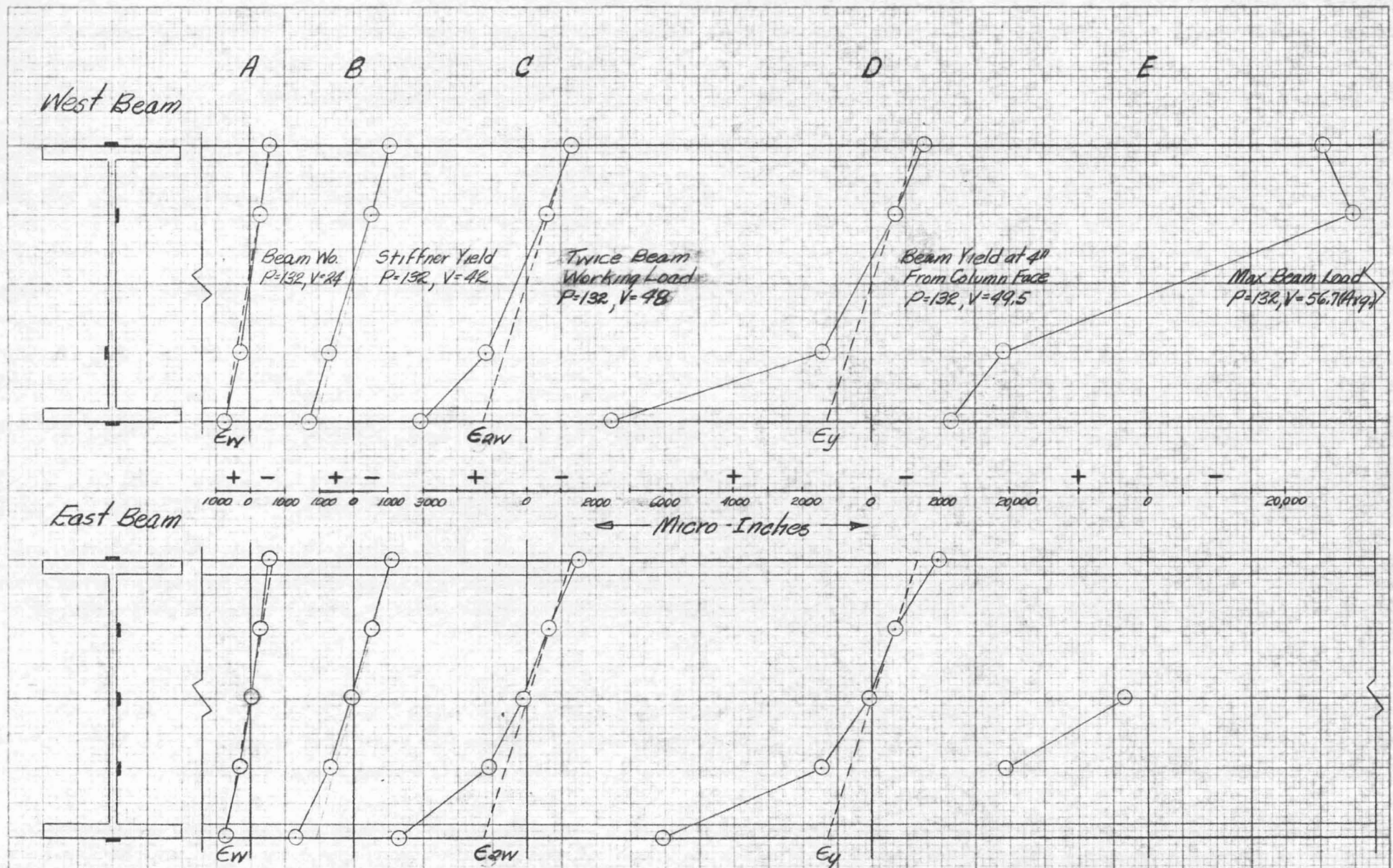


Fig. 14 Comparison Between Theoretical And Measured Strains In The 16W36 Beams,
At Various Beam Loads, At A Section 4-in. From The Column Face

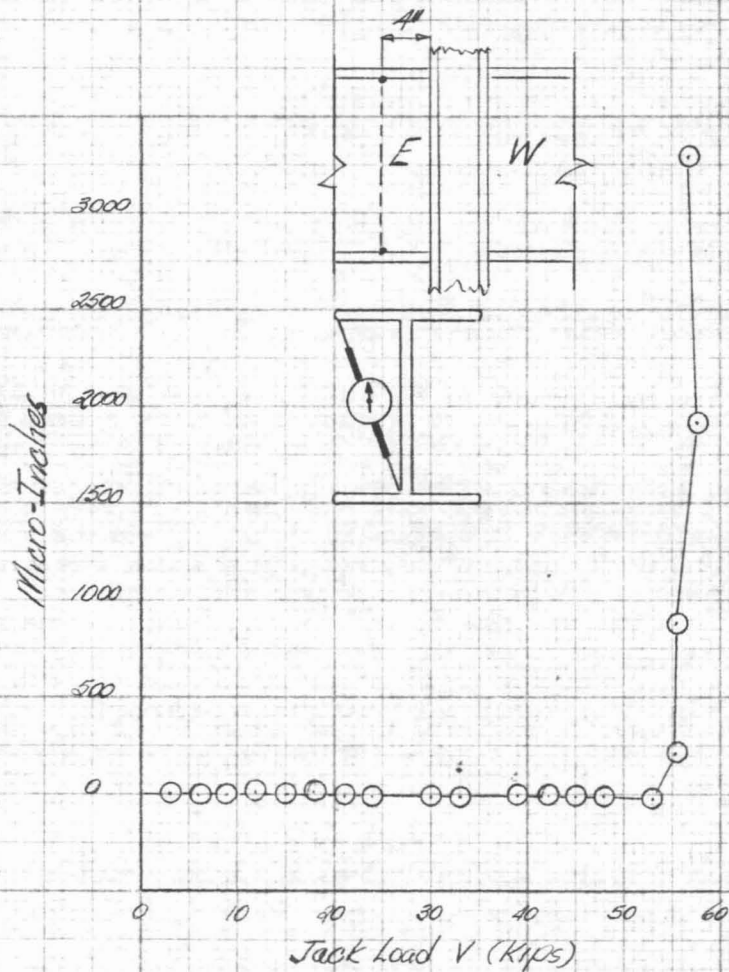


Fig. 15 Flange Buckling Curve At Point 8a,
On The West Beam

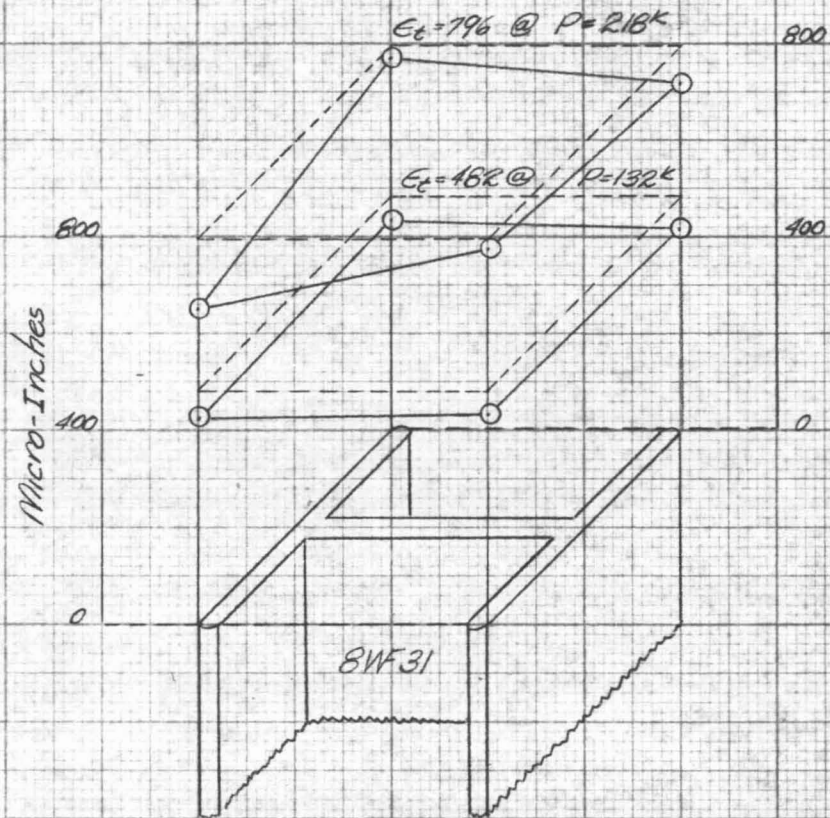


Fig. 16 Strain Distribution In Column
4 in. Above The Compression Beam Flanges

curve by showing that both beam failures were indeed due to flange and web buckling of the beams themselves. The stiffeners can be seen to be undamaged. Web buckling was not measured due to an oversight, but it was carefully observed while the test progressed.

(e) Whitewash

Whitewash was applied to the entire specimen and was used to indicate conditions of local yield. Strain patterns may be observed in each of the preceding photos of the specimen. There is a set of 8 color slides of the specimen made during the test on file in Fritz Laboratory. They show whitewash patterns while the test was in progress.

2. Column

(a) Strain Distributions

Although only a 9 foot column was used for the test specimen, the working load is determined on the basis of a 12' column. As stated before, actual conditions were simulated. Since the interest did not lay in the behavior of the column, but rather in the connection, a 12' column length would have been wasteful of materials. This method produced stresses at the connection corresponding to the working load of a 12' column with a saving in material but without a significant restriction of lateral deflection of the column had it been inclined to do so.

A strain distribution through the column x-section 4" above the compression beam flanges is shown in Fig. 16 for both the column working load and overload ($1.65 P_w$). The experimental strains compare favorably with theoretical values. The experimental strains are also quite uniform indicating axial load.

(b) Lateral Buckling

A dial mounted on the testing machine and reading deflections at the midpoint of a tension stiffener was used to check on column buckling. A plumb bob was used as a visual aid. Fig. 17 shows the load-deflection curve for the column. The maximum deflection which occurs is only 0.35". Neither the column or the beams were laterally supported at any point. When one considers the large deformations that were induced in the beams in a symmetrical manner so that the beams worked together to rotate the column (see Fig. 11), the absence of a significant column deflection is somewhat of a surprise.

3. Connection

(a) Stiffeners

As previously described, the stiffeners, together with the column web form a beam through the column. The use of stiffeners undoubtedly "makes" the connection. The proof of this statement can be seen in Fig. 11. Test #1 ended in a column web failure. Neither the column web or the stiffeners showed any signs of distress in test #2.

The strain distributions through the stiffeners is shown in Fig. 18. The curves indicate uniform distributions to the yield point (A and B), after which the strains become irregular (C and D). A similar set of curves is shown in Fig. 12. These give a direct comparison of strain distributions for both beams and the stiffeners on the compression side. Fig. 19 shows curves for the 8 gage average strain for the tension and compression stiffeners. These curves also demonstrate the consistency of the greater magnitude of the strains in the tension side of the specimen. The compressive strains indicate a slope

Beam Load V in Kips

Deflection in Inches

Column Buckling

Column load increased from 132k to 218k and brought back to 132k

16WF36

24-BWF31

132k Constant

P_u

Fig. 17 Load-column buckling curve.

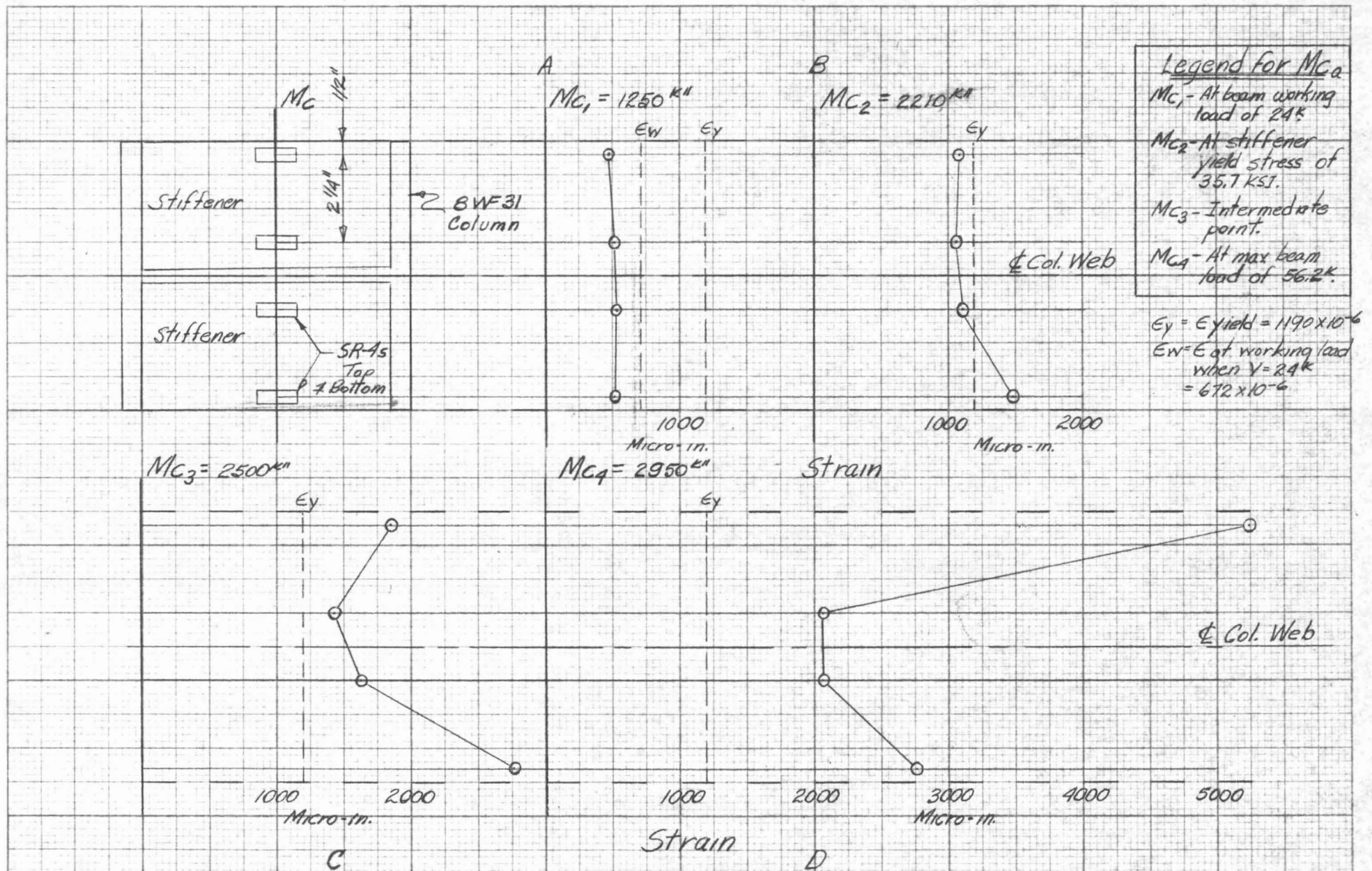


Fig. 18 Strain Distributions Through Compression Stiffeners
At Significant Column Centerline Moments

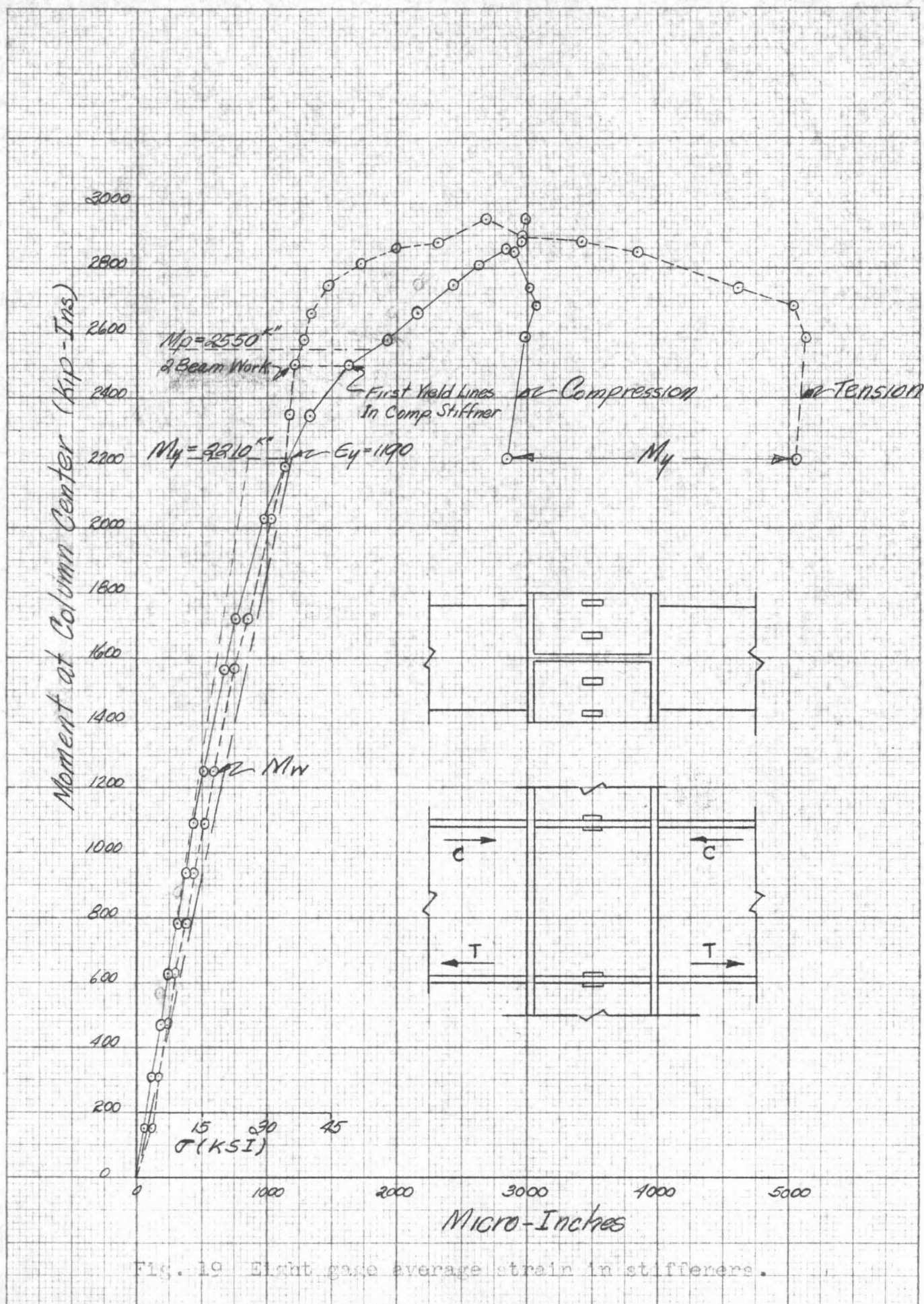


Fig. 19 Eight gage average strain in stiffeners.

steeper than the theoretical curve. This is due to web participation above that considered in determining I for the connection "beam".

(b) Column Web

The determination of web participation is accomplished most directly by calculating it from web strain distributions. These strains are shown in Fig. 20 as recorded by the horizontal legs of 45° rosetts that were placed along the centerline on the column web as shown. The use of these curves is fully explained in later discussions.

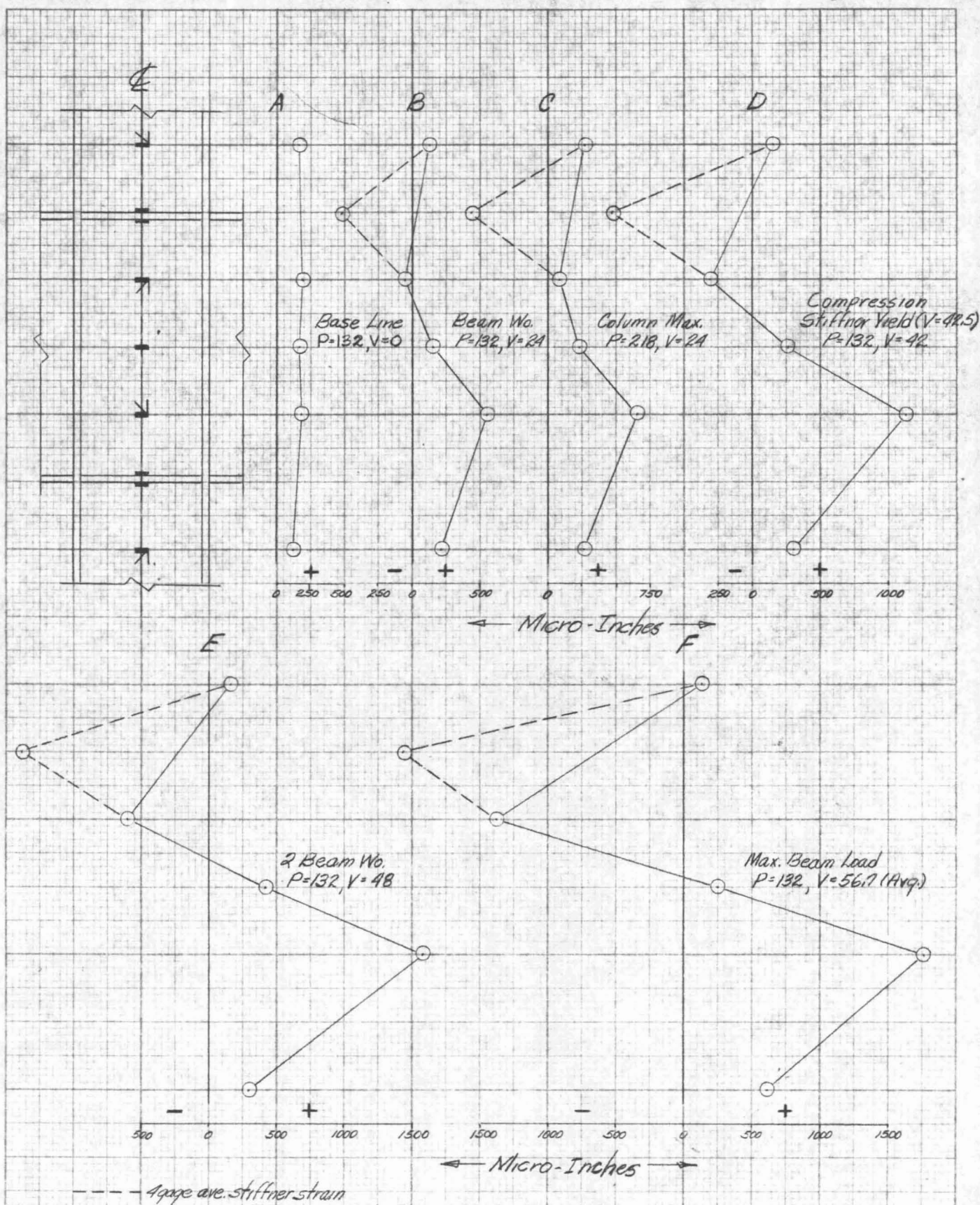


Fig. 20 Measured strains in the column web.

V. DISCUSSION

1. Beams

The striking feature of the test was the strength of the beams. The plastic strength of the beams was calculated to be 51.1 kips. Allowing about a 20% reduction for stress concentrations developed in rolling and welding the beam to the column, a maximum jack load of 40 to 45 kips was expected. The actual jack load exceeded the calculated plastic strength by better than 10% for each beam. After maximum load was reached, there was no sudden drop in the load carrying capacity, but rather a gradual decline. At the final test position, the beam was carrying 94% of yield load with an end deflection of 52 y. The final load readings can be compared only with respect to deflection since the rotation readings had to be discontinued when the buckled compression flanges touched the rotation bars.

Strain distribution readings at 4" from the column flange (Fig. 14) indicate that the rotation was not the result of a conventionally assumed strain distribution. The tension strains for the larger moments exceed those in the compression flange even though buckling had not as yet taken place. It appears that after the yield moment was reached, the N. A. gradually moved toward the compression flange. As can be seen in Fig. 13 the strain distribution was definitely non-uniform in the tension flange. Close inspection of the welds did not reveal any yielding or tearing of welds joining the column to the tension flanges. The lack of a uniform strain distribution appears to be an inherent shortcoming of the steel material and

its shape rather than the result of external conditions. Had the strain distribution been more uniform through the tension flange the beam deflections would undoubtedly be smaller.

The whitewash pattern indicated longitudinal tension strains along the tension flange by means of a clear pattern of 45° shear planes in the vicinity of the weld. The pattern was very heavy within the first 3" from the column flange, indicating clearly that this was the region of maximum yield. The initial yield lines were reached for this region.

Flange buckling was both closely observed and measured. Each compression beam flange buckled at about 3 kips below the ultimate load the beam attained. The ultimate loads were only 1.6 kips apart. Both beams had exceeded their calculated plastic strength before they buckled. Buckling occurred at $> 5 \sigma_y$ and $11 \delta_y$, indicating the region of buckling was in a plastic condition. Web buckling was not measured, but it was observed. For each beam web buckling was observed at the load increment following the initial flange buckling of the beam.

2. Column

The column did not show any signs of distress throughout the test. The curves in Fig. 16 indicate that axial load was maintained throughout the test. Axial load is usually assumed and not given much consideration. Because both beams buckled symmetrically so as to create a joint twisting effort, an eccentricity of column load in the direction of twist in this test could have caused a column failure. In an actual structure the presence of unequal beam loads can further reduce

the column capacity. The conditions of beam buckling though, are rarely if ever approached in practice.

3. Connection

(a) Stiffeners

When it is determined that stiffeners are needed, the question of how much stiffening is required arises. The stiffeners in this test were stressed well beyond their yield strength, but showed no signs of failure. The theoretical jack load at stiffener yield is 42.5 kips. Yield lines on the compression stiffeners were first observed at 48 kips. Strain distributions in the stiffeners were used to determine the average strain in the stiffeners. The maximum average strain was almost 3 times the yield strain. By the yield strength design criterion both the tension and compression stiffeners were failed. In order to form some basis for stiffener size evaluation, the amount of web participation is evaluated using stiffener strain distributions.

The beam strain distributions are used to determine what is coming into the connection. The stiffener strains indicate what share of the load the stiffeners are carrying. The difference is the web participation. Web participation is calculated using this method for jack loads of 24, 33, and 42 kips; corresponding to the beam working load, an intermediate load, and stiffener plate yield load (See Appendix II for calculations). A value of about $1/3$ is indicated. This value is also checked by methods still to be presented. The principle aim here is to try to correlate the percent of web participation indicated by strain distributions to the percent

of moment of inertia contributed by the web to the acting cross section is determined from the slopes of experimental curves.

(b) Web

The importance of knowing the amount of web action has already been emphasized. Web action was also analyzed through the use of SR-4's mounted on the web. The strain distributions indicated by the curves in Fig. 20 are used to determine the average stress in the column web. The load resisted by the web is expressed as a percentage of the load applied by the beam. As in all previous calculations, only the compressive strains are used because the tension strains were extremely large and often erratic. For beam loads of 24, 33, and 42 kips an average value of 35 percent was obtained. Again roughly $1/3$ (See Appendix II).

The depth of web action can be directly estimated from the $M-\phi$ curve for section 4-4 and the 8 gage average strain curves for the stiffeners. The I value for the actual curve is computed. The I value of the stiffeners alone is subtracted. The difference represents the I of the web. The "c" value in making this approximation is one half the over-all depth of the stiffener. This is not strictly correct. The $M-\phi$ curve indicates a web moment of inertia of 32% while the 8 gage average strain curve suggests 38%. It must be remembered that these values are based on the slopes of the curves which are subject to individual interpretation and the approximate "c" value of $1/2$ over-all stiffener depth.

(c) Welds

The welds on the specimen showed no signs of failure except for one spot. The beam web to column flange weld on the West beam started to tear at the flame cut in the tension side of the beam. The maximum beam load had just been reached and the next increment of deflection was being applied when a loud knock was heard. Examination of the specimen revealed the tear. It did not become worse as the test progressed though. All other welds remained sound throughout the test, proving they were just as strong as the parent material.

4. Design Recommendations

The method of a design criterion for a stiffened connection must account for many variables. These include beam depth, web thickness, and flange thickness; column depth and web thickness; and beam end moment. One test cannot be considered adequate background for making any kind of recommendation, but a procedure will be presented based on this test.

This test suggests the use of the following procedure for stiffeners that will be placed at the same level as the beam flanges:

1. Determine d_e by increasing beam d by 25%. This represents the extent of web participation.
2. Determine web I_w using web thickness and d_e by using $I = 1/12 t_w d_e^3$.

3. Determine necessary total I_t to resist the moment developed at the center of the connection using an allowable stress and $c = 1/2 d$.

4. Stiffener $I_s = I_t - I_w$

5.
$$t_s = \frac{2 I_s}{(b-t_w)d^2}$$

where b = column flange width

d = beam depth

t_s = stiffener thickness

t_w = column web thickness.

This approach may be an over simplification of the problem, but it uses the basic ideas which test #2 suggests.

The same thickness of stiffeners for both the tension and compression sides is recommended. The same method of fabrication as was used in this test is also suggested.

VI. SUMMARY AND CONCLUSIONS

The statements that follow are based on the results of this single test.

1. Connection Design

1. The stiffeners for this connection develop approximately $2/3$ of the connection I, based on acting web area.
2. The acting web area extends about $.125d$ beyond the stiffeners on each side.
3. The stiffeners resist approximately $2/3$ of the beam end moments developed in this connection.
4. The stiffener thickness for a similar connection can be determined on the basis of total I (stiffener I + web I) needed to resist beam end moment.
5. The stiffener will not fail for the above stiffener thickness.
6. The full rolled section strength is developed in the welds at the column flange.
7. The full beam strength is developed for this connection.
8. Column working load does not affect the strength of the connection.
9. Column overload is not detrimental at beam working load.

2. Structural Behavior

1. First yield in the beams (whitewash) occurred at 53% of computed initial beam yield load.
2. The beams developed greater moments than the predicted theoretical plastic moment.
3. The critical beam sections occur at about 6" from the column face.
4. Once the predicted yield load was reached, the end deflections increased rapidly.
5. The average unit rotation for section 1-1 was excessive because of stress concentrations due to welding.
6. Large flame cuts of beam web are undesirable. The tension side of one beam web started to tear loose.
7. Structural behavior will be similar for connections stiffened in the recommended manner for the commonly used sizes of wide flange sections.

A C K N O W L E D G E M E N T S

This report has been prepared as a result of research carried out at Fritz Engineering Laboratory under the supervision and guidance of Dr. Lynn S. Beedle, assistant director of Fritz Engineering Laboratory. Prof. Cyril D. Jensen and research assistant Archibald N. Sherbourne contributed generously of their time and made valuable suggestions. The author also wishes to express his appreciation to those members of the Lehigh research staff who assisted in the testing.

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N O M E N C L A T U R E

A	=	Area of section
A_s	=	Area of stiffeners
A_w	=	Area of web
b	=	Flange width
c	=	Distance from neutral axis to extreme fiber
d	=	Depth of section
d_e	=	Effective depth of web
δ_y	=	Beam end deflection at yield
E	=	Young's modulus of elasticity
ϵ_w	=	Unit strain at working load
ϵ_{2w}	=	Unit strain at twice working load
ϵ_y	=	Unit strain at yield
I_s	=	Stiffener moment of inertia
I_t	=	$I_s + I_w$
I_w	=	Web moment of inertia
K	=	Distance from outer face of flange to web toe of fillet
L_f	=	Load at flange buckling
L_y	=	Load at beam yield
M_b	=	Allowable beam end moment
M_p	=	Plastic moment
M_r	=	Resisting moment
M_y	=	Yield moment
N	=	Length of bearing of beam flange on column
NA	=	Neutral axis
P	=	Column load
P_w	=	Column working load
R	=	Concentrated load on column flange

ϕ_y = Average unit rotation at yield
 S = Section modulus
 σ_o = Allowable plate stress
 σ_{cr} = Critical plate stress
 t_b = Beam web thickness
 t_w = Column web thickness
 t_s = Stiffener web thickness
 V = Beam load
 V_w = Beam working load
 Z = Plastic section modulus

APPENDIX I

Calculations for Section Properties

1. 16 WF 36 Beam

$$d = 15.35" \quad w_f = 6.992" \quad t_f = .428" \quad t_w = .277" \quad I_{xx} = 446.3"{}^4 \quad A = 10.59"{}^2$$

$$\sigma_y = 38.9 \text{ KSI} \quad E = 30,000 \text{ KSI} \quad S = 56.3"{}^3$$

A. Elastic Properties

$$M = \sigma S$$

$$M_y = 38.9 \times 56.3 = 2190 \text{ K}\cdot\text{in}$$

$$\phi = M/EI$$

$$\phi_y = 2190 / 30,000 \times 446.3 = 163.5 \times 10^{-4} \text{ in}^{-1}$$

$$\epsilon = \sigma/E$$

$$\epsilon_y = 38.9 / 30,000 = 1295 \times 10^{-6} \text{ in/in}$$

$$\delta = PL^3/3EI$$

$$\delta_y = 45.6 \times 48^3 / 3 \times 30,000 \times 446.3 = .126"$$

B. Plastic Properties

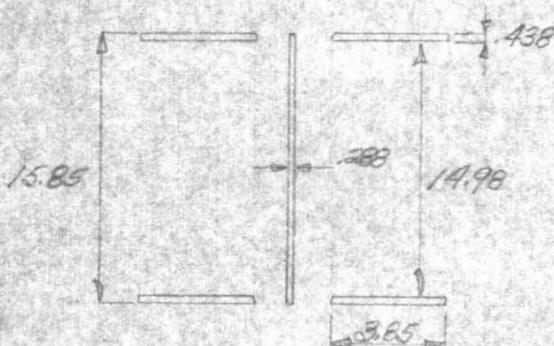
$$Z = 2 \left[(6.99 \times .428) 771 + .277 \times \frac{(7.495)^2}{2} \right]$$

$$= 46.25 + 16.82 = 63.1"{}^3$$

$$M_p = \sigma_y \times Z_p$$

$$= 38.9 \times 63.1 = 2455 \text{ K}\cdot\text{in}$$

2. Stiffened Section



$$I = 4 \times 3.85 \times .438 \times (7.72)^2 + 1/12 \times .288 (15.85)^3$$

$$= 401 + 92 = 493"{}^4$$

$$Z = 4 \times 3.85 \times .438 \times 7.72 + 2 \times 7.94 \times .288 \times 3.87$$

$$= 52.0 + 18.15 = 70.15"{}^3$$

$$\sigma_y = 35.7 \text{ KSI} \quad E = 30,000$$

Section properties cont'd

A. Elastic Properties

$$M = \sigma I / c$$

$$M_y = 35.7 \times 493 / 7.94 = 2210 \text{ k-in}$$

$$\Phi = M / EI$$

$$\Phi_y = 2210 / 30,000 \times 493 = 150 \times 10^{-4} \text{ in}^{-1}$$

$$\epsilon = \sigma / E$$

$$\epsilon_y = 35.7 / 30,000 = 1190 \times 10^{-6} \text{ in/in}$$

B. Plastic Properties

$$M_p = \sigma_y \times Z$$

$$= 35.7 \times 70.15 = 2550 \text{ k-in}$$

3. Columns

$$\text{Area} = 9.12 \text{ in}^2 \quad E = 30,000 \text{ ksi}$$

$$\text{Col. working load for 12' col.} = 132 \text{ k}$$

$$\sigma = \frac{P}{A} = \frac{132}{9.12} = 14.45 \text{ ksi}$$

$$\epsilon_w = \frac{\sigma}{E} = \frac{14.45}{30,000} = 482 \times 10^{-6} \text{ in/in}$$

$$\epsilon_{AL} = 1.65 \times 482 \times 10^{-6} = 796 \text{ in/in}$$

4. Special Calculation

The results of beam coupon tests using an automatic stress-strain recorder indicated a beam E value of 25,500 ksi. These tests were performed by A.N. Sherborne. It was felt that the automatic stress-strain recorder was not precise enough and the test value was disregarded upon Mr. Sherborne's advise. The value of ϵ_y obtained

by using an E of 25,500 ksi is indicated on Fig. 12 as ϵ_y' only as a means of comparison. It is not used in any of the calculations.

$$\epsilon_y' = \frac{38.9}{25,500} = 1525 \times 10^{-6} \text{ in/in.}$$

It can be seen in Fig. 12A that at the beam working load the measured strains agree quite well with the theoretical, calculated by using an E of 30,000 ksi; and are, in fact, slightly less than the theoretical value. Fig. 12B shows measured strains at the stiffener yield load which is 72 kips less than the beam yield load for the section 4" from the column face. The measured strains fall short of ϵ_y based on $E=30,000$ ksi, as they should, showing a very good correlation between measured and theoretical strains. The use of $E=30,000$ ksi for the beams is based on values obtained by members of the Fritz Laboratory research staff for similar sections.

APPENDIX II

Web Participation Calculations

1. Estimate based on stiffener & beam strain distributions.

The basis for these calculations is that the whole is equal to the sum of its parts. The strain gages on the East & West beams are used to determine what is going into the column connection on the compression side. Knowing the size of the stiffener & the strain distribution at the center of the stiffener, the proportion of the load taken by the stiffener may be determined. The difference between the total or beam load & the stiffener is necessarily taken by the web either by direct bearing or transmission through the web-stiffener welds.

The beam gages are located 44" from the load point while the stiffener gages are at 52". Obviously, for any given load, the gages for the stiffeners & the beams are recording for different moments. A moment arm ratio is used to multiply the beam gage readings to equate moment conditions. The assumption that most of the beam compressive thrust is applied thru the compression flange is made. A further beam gage reading modification is made to compensate for the difference

Web participation cont'd

between the beam flange area & stiffener area. In all cases as many gages as are pertinent to the average strain reading sought are used to obtain better accuracy.

A. SR-4 Data

(1) Beams

Ewb (V=24K)		Eys (V=42K)		EI (V=35K)	
East	West	East	West	East	West
78 559	40 539	78 989	40 1149	78 785	40 799
79 518	41 566	79 1029	41 1032	79 730	41 818
81 541	42 612	81 1110	42 1202	81 798	42 862
82 548	43 745	82 1084	43 1462	82 795	43 1056
Ave 541	Ave 615	Ave 1053	Ave 1211	Ave 779	Ave 884
2 Em. Ave = 578		2 Em. Ave = 1132		2 Em. Ave = 832	

(2) Stiffeners (Compression)

	54	55	56	57	62	63	64	65	Ave
Ewb	494	486	470	526	523	500	518	530	509
Eys	1026	1171	936	998	1065	1480	1337	1041	1131
EI	742	741	713	768	772	716	798	790	755

B. Computations:

$$\text{Area beam flg} = 0.438 \times 6.97 = 3.04 \text{ in}^2$$

$$\text{Area comp. stiff.} = 2 \times 7/16 \times 3.85 = 3.37 \text{ in}^2$$

$$\text{Moment arm ratio} = \frac{52}{44} = 1.181$$

$$E = 30,000 \text{ ksi}$$

(1) Ewb

$$T_b = 1.181 \times E \times \epsilon \times A \text{ or } T_s = E \times \epsilon \times A$$

where T_x = Thrust of beam or stiffener

ϵ = gage ave. strain for beam flg or stiffener

$E = 30,000 \text{ ksi}$

A = area of beam flg. or stiffeners

Web participation cont'd

$$T_b = 1.181 \times (576 \times 10^{-6}) \times 30,000 \times 2.99 = 61.3^\circ\text{K}$$

$$(T_b = \text{const.} \times 1/\text{in} \times K/\text{in}^2 \times \text{in}^2 = K)$$

$$T_s = (509 \times 10^{-6}) \times 30,000 \times 3.37 = 51.5^\circ\text{K}$$

$$T_w = T_b - T_s = 61.3 - 51.5 = 9.8^\circ\text{K}$$

$$\% \text{ Part. Web} = 9.8 \div 61.3 \times 100 = 16.0\%$$

(2) E_I

$$T_b = 1.181 \times (832 \times 10^{-6}) \times 30,000 \times 2.99 = 86.3^\circ\text{K}$$

$$T_s = (755 \times 10^{-6}) \times 30,000 \times 3.37 = 76.4^\circ\text{K}$$

$$T_w = 86.3 - 76.4 = 9.9^\circ\text{K}$$

$$\% \text{ Part. Web} = 9.9 \div 86.3 \times 100 = 11.5\%$$

(3) E_{ys}

$$T_b = 1.181 \times (1132 \times 10^{-6}) \times 30,000 \times 2.99 = 120.2^\circ\text{K}$$

$$T_s = (1131 \times 10^{-6}) \times 30,000 \times 3.37 = 114.5^\circ\text{K}$$

$$T_w = 120.2 - 114.5 = 5.7^\circ\text{K}$$

$$\% \text{ Part. Web} = 5.7 \div 120.2 \times 100 = 4.7\%$$

(4) Summary

E	V	$\%$
E_{wb}	24	16.0
E_I	33	11.5
E_{ys}	42	4.7

Web participation cont'd

(5) The previous calculations did not take the beam web into consideration. We will now do so.

(a) SR-4 Data

	$E_{wb} (V=24)$	$E_{ys} (V=42)$	$E_r (V=33)$
East (41)	566	1032	818
West (82)	548	1089	883
	557	1058	850

(b) Computations

1. E_{wb}

$$Tw' = \frac{E}{2} \times E \times t_w \times d/r \quad \text{where } Tw' = \text{beam web load}$$

$$= \frac{557 \times 10^{-6} \times 30,000 \times .999 \times 7.92}{2} = 19.8 \text{ k}$$

$$\% \text{ Web Part.} = \frac{9.8 + 19.8}{61.3 + 19.8} \times 100 = 36.4\%$$

2. E_r

$$Tw' = \frac{850 \times 10^{-6} \times 30,000 \times .999 \times 7.92}{2} = 30.2 \text{ k}$$

$$\% \text{ Web Part.} = \frac{9.9 + 30.2}{86.3 + 30.2} \times 100 = 34.4\%$$

3. E_{ys}

$$Tw' = \frac{1058 \times 10^{-6} \times 30,000 \times .999 \times 7.92}{2} = 37.6 \text{ k}$$

$$\% \text{ Web Part.} = \frac{5.7 + 37.6}{120.2 + 37.6} \times 100 = 27.4\%$$

4. Summary

E	V	%
E_{wb}	24	36.4
E_r	33	34.4
E_{ys}	42	27.4

Web participation cont'd

2. Estimate based on column web strain distributions.

SR-4 gages placed along the center line of the column web in the connection region are here used to determine stress distributions in the compression portion of the web. The average web stress \times the compressive area of the web determines the web participation.

The SR-4 information is very limited for only three gages were actually located on the column web in the compressive region (one at the N.A.). A fourth point is obtained by averaging the 4 SR-4s on the stiffeners that are adjacent to the web. In each case in the elastic range (as determined by compression stiffener yield lines) the 4 gages record strains very close to one another. Since the stiffeners are welded to the web, it is logical to assume that the immediate area, including the web, behaves in the same manner.

Referring to Fig. 20 it is observed that the web is initially ($P=132^k$, $V=0^k$) in tension due to the Poisson Ratio. These points lie in an approximate straight line. This line is used as

Web participation cont'd

the base and the strains due to beam load are calculated relative to it. It should be noted that due to the off loading of the lower part of the column ($132^k - 2V$) this base line is not valid for the tension region of the connection. The compressive region will undoubtedly be affected somewhat, but the effect is neglected because of gross approximations that are made regarding the actual strain pattern. A triangular stress pattern is assumed although the true stress pattern probably approximates a parabolic distribution in the vicinity of the stiffeners. If all 4 gage points have a compressive value, the compressive area is assumed to be $1/2$ the beam depth plus $3\frac{1}{2}"$ (See Fig 20.). When a tension reading occurs at a point, the compressive area is computed.

A. SR-4 Data

	$P=132, V=0$	$E_{wb}(V=20)$	$E_{ls}(V=40)$	$E_t(V=33)$
97	156	118	151	169
$(34+57+62+65)/4$	15	-518	-1033	-768
96	105	-51	-305	-135
93 (N.A.)	170	150	265	198

Web participation cont'd

B. Computations

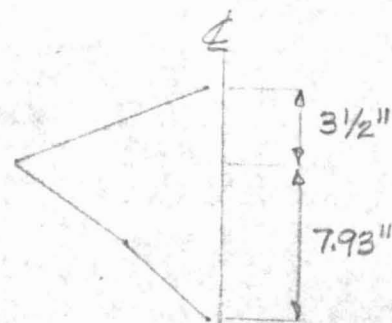
$$\text{Web thickness} = 0.299''$$

$$\text{Web depth} = 15.85''$$

$$E = 30,000 \text{ KSI}$$

WEwb

97	$156 - 118 = 38 \text{ C}$
Stiff.	$15 - (-518) = 533 \text{ C}$
96	$195 - (-51) = 246 \text{ C}$
93	$170 - 150 = 20 \text{ C}$



(a)*

As can be seen in sketch (a) a triangular distribution is approximated.

$$A_{\text{comp}} = 0.299(3.5 + 7.93) = 3.12 \text{ in}^2$$

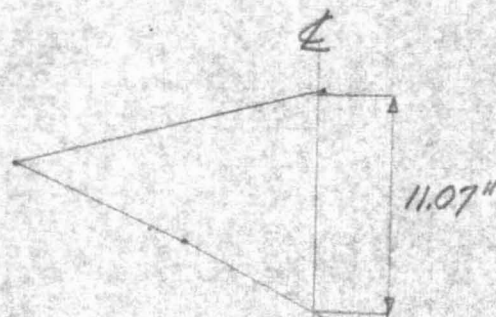
$$\begin{aligned} \text{Web Part.} &= \frac{E}{2} \times E \times A \\ &= \frac{533 \times 10^{-6} \times 30,000 \times 3.12}{2} = 27.4 \text{ K} \end{aligned}$$

% web participation is based on beam thrust as calculated previously (Section 1).

$$\% \text{ Web Part} = 27.4 \div 61.3 = 44.6\%$$

(2) EI

97	$156 - 169 = -13 \text{ T}$
Stiff.	$15 - (-768) = 783 \text{ C}$
96	$195 - (-135) = 330 \text{ C}$
93	$170 - (198) = -28 \text{ T}$



(b)*

Web participation cont'd

$$\frac{13}{783+13} \times 3.50 = .05"$$

$$\frac{28}{330+28} \times \frac{7.93}{2} = .31"$$

$$A_{comp} = 0.299(11.43 - .36) = 3.31 \text{ in}^2$$

$$\text{Web Part.} = \frac{783}{2} \times 10^{-6} \times 30,000 \times 3.31 = 38.9\%$$

$$\% \text{ Web Part} = 38.9 \div 86.3 \times 100 = 45\%$$

(3) E_{ys}

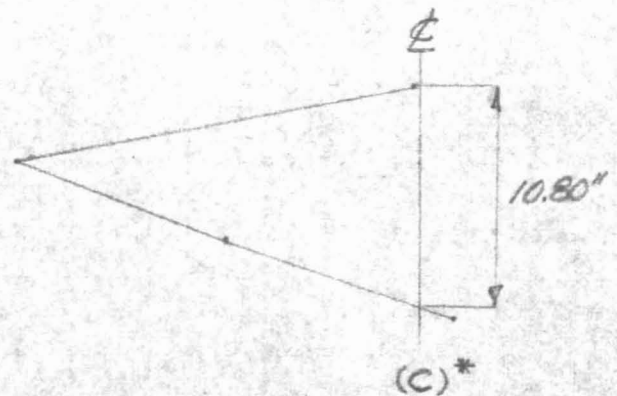
$$\begin{array}{lll} 97 & 156 - 151 & = 5C \\ \text{Stiff.} & 15 - (-1083) & = 1048C \\ 96 & 125 - (-305) & = 500C \\ 93 & 170 - 265 & = -95T \end{array}$$

$$\frac{95}{500+95} \times \frac{7.93}{2} = .63"$$

$$A_{comp} = 0.299(11.43 - .63) = 3.23 \text{ in}^2$$

$$\text{Web Part.} = \frac{1048}{2} \times 10^{-6} \times 30,000 \times 3.23 = 50.8\%$$

$$\% \text{ Web Part} = 50.8 \div 120.2 \times 100 = 42.3\%$$



(4) Summary

E	V	%
E _{ub}	24	44.6
E _r	33	45.0
E _{ys}	42	42.3

* Horizontal Scale: 1" = 500 x 10⁻⁶ in/in

Web participation cont'd

(5) Considering the beam web.

(a) E_{wb}

$$\% \text{ Web Part.} = 27.4 \div (61.3 + 19.8) \times 100 = 33.8\%$$

(b) E_I

$$\% \text{ Web Part.} = 38.9 \div (86.3 + 30.2) \times 100 = 33.4\%$$

(c) E_{ys}

$$\% \text{ Web Part.} = 50.8 \div (120.2 + 37.6) \times 100 = 32.2\%$$

(6) Summary

	E	V	%
E_{wb}	24		33.8
E_I	33		33.4
E_{ys}	42		32.2

3. A comparison of values obtained.

The average web participation, as determined by the strain distributions in the stiffener, is 32.7%; while that obtained by using the column web strain distributions is 33.1%. The values indicate that for this connection, the web resisted roughly $\frac{1}{3}$ of the compressive force that was placed on it by the beams.

Extent of Web Participation

It has been previously pointed out that the extent of web participation is more than that enclosed by the stiffeners. This extent can be estimated directly from two previous curves, namely the $M-\phi$ curve for section 4-4 and the 8 gage average strain curve for the compression stiffener.

(1) Eight Gage Average Compressive Strain

In this case the stiffener strain is related to the continuous beam I as shown below:

$$\sigma = \frac{Mc}{I}, \text{ also } \epsilon = \frac{\sigma}{E}$$

$$\text{hence } \epsilon = \frac{Mc}{EI}$$

From the slope of the actual curve a new ϵ_y is obtained as shown in Fig. 18. With this ϵ a new I may be computed.

$$E = 900 \times 10^6 \text{ in/in}$$

$$I = \frac{3210 \times 7.98}{30,000 \times 900 \times 10^6}$$
$$= 648 \text{ in}^3$$

I for stiffeners alone is 402 in^3

$$\therefore \frac{1}{2} \Delta I = 648 - 402 = 246 \text{ in}^3$$

$$\Delta I = \frac{246 \times 12}{.788}$$

$$d_e = 23.20"$$

The d value of 23.20" indicates a web participation depth as extending 3.7" beyond the stiffeners. The preceding calculations are not strictly correct because the c value is for the outer side of the stiffener. The calculation is based on compressive strains only, because the compressive values were more consistent. The tension values indicate excessive deformations.

(2) $M-\phi$ Curve for Section 4-4

The average unit rotation at section 4-4 is related to the continuous beam I in the following manner:

$$\phi = \frac{M}{EI}$$

From the slope of the experimental curve a new ϕ_y is obtained as shown in Fig. 9. With this new ϕ_y a new I may be computed.

$$\phi_y = 1.24 \times 10^{-4} \text{ in}^{-1}$$

$$I = \frac{2210}{30,000 \times 1.24 \times 10^{-4}}$$

$$= 594 \text{ in}^3$$

$$\therefore \frac{1}{12} t d^3 = 594 - 402 = 192 \text{ in}^3$$

$$d^3 = \frac{192 \times 12}{.288}$$

$$d_e = 19.95"$$

The horizontal strain gages on the column web centerline indicated that the web participation was only very slight at $3\frac{1}{2}$ inches beyond the stiffeners. The d_e value of 23.20" as determined from the slope of the 8-gage average stiffener curve indicates in theory full web participation to 3.67 inches beyond the stiffener or partial participation for an even greater distance beyond the stiffener. This d_e value is obviously too large. The d_e value determined from the slope of the $M-\Phi$ curve is 19.95 inches, and it is a realistic value. The effective web depth d_e is therefore estimated to be approximately $1.25 \times$ the beam depth. A d_e for this connection would include 2 inches beyond the stiffeners if d_e is assumed to be $1.25d$. The d_e , it must be remembered, is a calculated value with no corresponding point on the column web. The actual d_e is, no doubt, greater than $1.25d$ but the web does not act fully throughout. This actual d_e partially acting, has the same effective I value as the calculated d_e which is based on full section participation.

Stiffener Calculations

The expression for determining safe buckling load for a given plate thickness for the stiffener condition is:

$$\sigma_0 = \frac{t^2}{12(1-\nu^2)} \left[E_{tx} \left(\frac{\pi}{l} \right)^2 + E \left(\frac{2}{b} \right)^4 \left(\frac{l}{\pi} \right)^2 \frac{C_3}{C_2} - (E_{tx} + E) \nu \left(\frac{2}{b} \right)^2 \frac{C_5}{C_2} + 4(1-\nu^2) G_t \left(\frac{2}{b} \right)^2 \frac{C_7}{C_2} \right]$$

where $E = 30,000$ KSI

$E_{tx} = 900$ KSI

$G_t = 670$ KSI

$\nu = 0.30$

$C_2 = 0.04286$

$t = 7/16$ "

$C_3 = 0.56712$

$l = 7.13$ "

$C_5 = 0.0216$

$b = 8.00$ "

$C_7 = 0.17564$

Substituting these values into the preceding expression and solving we obtain:

$$\sigma_0 = \frac{0.438^2}{12(1-0.30^2)} \left[900 \left(\frac{3.141}{7.13} \right)^2 + 30,000 \left(\frac{2}{8.00} \right)^4 \frac{0.56712}{0.04286} \left(\frac{7.13}{3.141} \right)^2 - (900 + 30,000)(0.30) \left(\frac{2}{8.00} \right)^2 \frac{0.0216}{0.04286} + 4(1-0.30^2) 670 \left(\frac{2}{8.00} \right)^2 \frac{0.17564}{0.04286} \right]$$

$$\sigma_0 = 149 \text{ K}$$

Substituting the yield stress for σ_0 , a minimum thickness is obtained as follows:

$$t^2 = \frac{25.7 \times 12(1-30^2)}{\left[900 \left(\frac{3.141}{7.13} \right)^2 + 30,000 \left(\frac{2}{8.00} \right)^4 \frac{0.56712}{0.04386} \left(\frac{7.13}{3.141} \right)^2 \right.} \\ \left. - (900 + 30,000)(30) \left(\frac{2}{8.00} \right)^2 \frac{0.0216}{0.04386} + 4(1-30^2) 670 \left(\frac{2}{8.00} \right)^2 \frac{0.17564}{0.04386} \right]$$

$$t = 0.214"$$

This indicates that a $1/4"$ plate would have accomplished the job of stiffening this connection with respect to buckling failure. It would prove interesting to check this value with a test of a specimen so stiffened.

It should be remembered that the above calculations are for a condition of buckling failure. Though no deformation of the $7/16"$ stiffeners used took place, the stiffeners were failed if one considers the initial yield stress as the governing design criterion.

A note of caution that was mentioned in the initial presentation of the above expression is worth repeating. The expression does not exactly apply to the condition of plates used to stiffen columns. The preceding calculations, therefore, should not be thought of as representing accurate buckling solutions for the stiffener plates.